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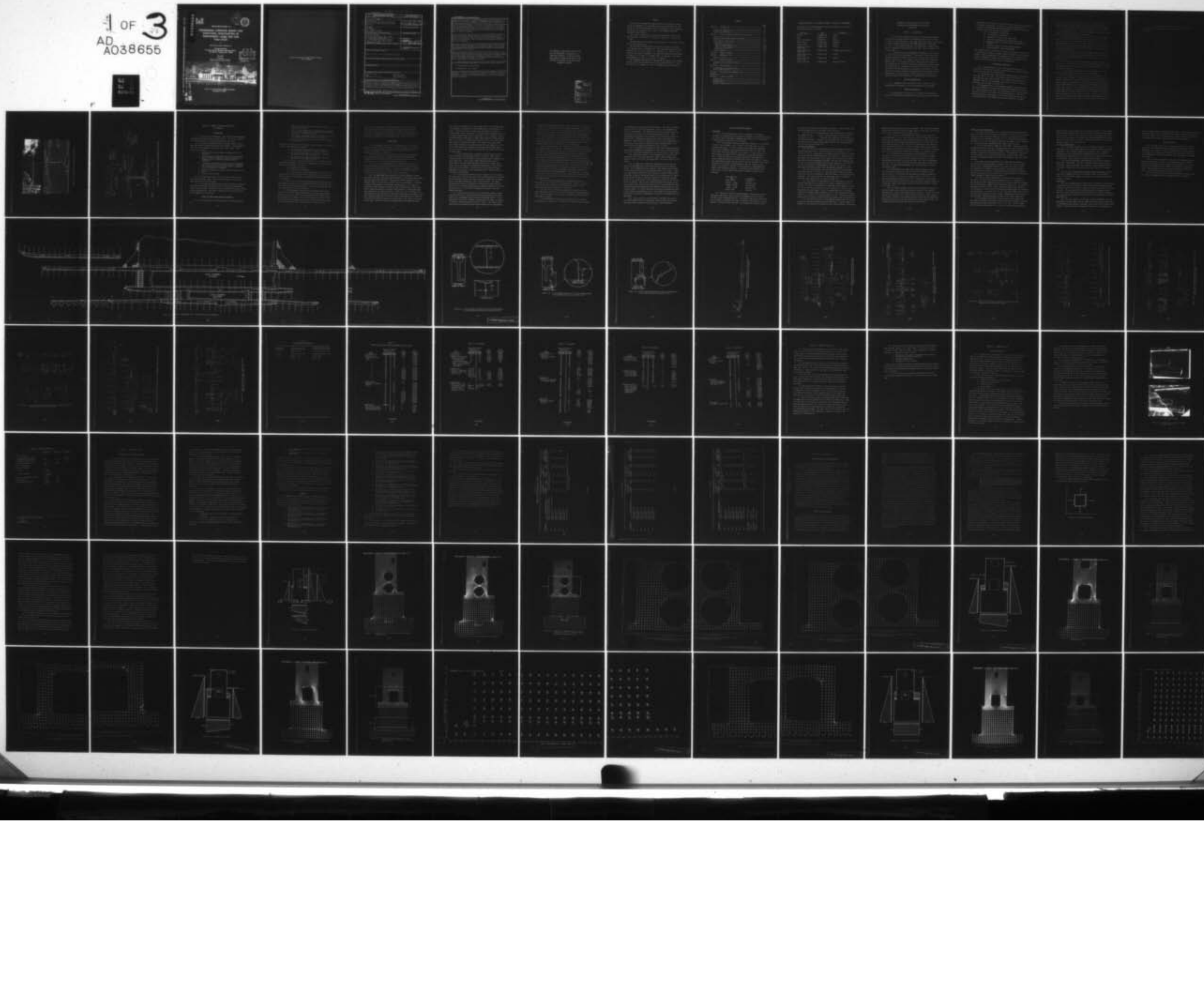

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ENGINEERING CONDITION SURVEY AND STRUCTURAL INVESTIGATION OF MONTGOMERY LOCKS AND DAM OHIO RIVER

by

Carl E. Pace, James T. Peatross, Jr.

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Final Report

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The Concrete Laboratory at the US Army Engineer Waterways Experiment Station was contracted to prepare an engineering condition survey and structural investigation for ~~Locks and Dam 3, Monongahela River, and Emsworth and~~ Montgomery Locks and Dams, Ohio River. This report gives the results of an engineering condition survey and a structural analysis of Montgomery Locks and Dams, Ohio River.

There is general spalling, leaching, and cracking of the concrete surfaces of Montgomery Locks and Dam. The crack survey implies that a majority of the

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cracking in the lock walls is caused by barge impact. The longitudinal crack parallel to the lock in the middle wall of Montgomery Lock is hypothesized to be caused by barge impact; therefore, this can be a source of deterioration which increases with lock use. The soniscope study indicates that the cracking along the center of the middle wall does not worsen with depth. The concern of the cracks and spalled areas, in the concrete surface, is that they will allow the access of water; thereby causing an increased rate of deterioration due to freezing and thawing. Maintenance of the surface cracks and spalled areas is, therefore, essential.

In relation to present-day criteria, almost all of the monoliths on the land wall are inadequate in their resistance to overturning and base pressures. In general, the monoliths in middle and river walls are inadequate in their resistance to overturning. This is especially true for the middle wall monoliths in the dewatered case. The miter sills are inadequate for sliding if the locks are dewatered.

The stress in the culvert wall of monolith M-8 is greater than 800 psi tension. This tensile stress is too large and will crack the concrete. This allows a stress flow up through the center of the monoliths, thereby causing cracking. This hypothesized condition for cracking should be checked by inspecting the culvert walls as soon as possible.

From the deteriorated condition of the surface of the lock monoliths, it is evident that some action must be initiated. Since corrective action is needed, a feasibility study should be made to determine what action is necessary which will provide the most economical and adequate lock usage over a period of 30 to 50 years. For this reason, it is recommended that a feasibility study be made considering the following alternatives:

a. Minimum maintenance and protection of the locks and dam from weathering with expected replacement when needed as determined by periodic inspection.

b. Rehabilitation of locks and dam.

c. Replacement of locks and dam.

The above recommendations may be affected by a total structural and operational evaluation. In fact, this study does not evaluate the foundation, steel gates, bridgework, lock gates, or appurtenant mechanical or electrical facilities. These will be considered by the Pittsburgh District in the overall evaluation of the locks and dam.

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PREFACE

The work of an engineering condition survey and structural investigation for Montgomery Locks and Dam located on the Ohio River was conducted for the US Army Engineer District, Pittsburgh, Corps of Engineers, by the Concrete Laboratory (CL) of the US Army Engineer Waterways Experiment Station (WES).

The contract was monitored by the Pittsburgh District Office with main assistance from Messrs. J. Colletti, H. Ferguson, J. Gribar, and S. Long.

The cooperation and assistance of all personnel at the District Office were greatly appreciated.

The study was performed under the direction of Messrs. B. Mather, J. M. Scanlon, and J. E. McDonald, CL. The structural analysis was performed by Dr. C. E. Pace, Messrs. R. L. Campbell, E. F. O'Neil, and J. T. Peatross, and Major H. Beardslee. The material properties were obtained by Messrs. R. L. Stowe, F. S. Stewart, and J. B. Eskridge. The report was prepared by Dr. Carl E. Pace and J. T. Peatross.

The Commanders and Directors of WES during the conduct of the program and the preparation and publication of this report were COL G. H. Hilt, CE, and COL J. L. Cannon, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, US CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

US customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	2.540000 E-02	metres
feet	3.048000 E-01	metres
miles (US statute)	1.6093 E+03	metres
pounds (mass)	4.535924 E-01	kilograms
pounds (force)	4.448222 E+00	newtons
pounds (force) per square foot	4.788026 E+01	pascals
pounds (mass) per cubic foot	1.601846 E+01	kilograms per cubic metre
pounds (force) per square inch	6.894757 E+03	pascals
kips (force) per square inch	6.894757 E+06	pascals
feet per second	3.048000 E-01	metre per second

ENGINEERING CONDITION SURVEY AND STRUCTURAL
INVESTIGATION OF MONTGOMERY LOCKS AND DAM
OHIO RIVER

SECTION 1: INTRODUCTION

1.1 This report contains the results of an engineering condition survey and a structural analysis of Montgomery Locks and Dams (Figures 1.1 and 1.2) on the Ohio River. The investigation was conducted from October 1974 to February 1976 by the Waterways Experiment Station (WES) for the U. S. Army Engineer District, Pittsburgh (ORP). Authorization for the investigation was given in DA Form 2544, dated 23 October 1974, issued by ORP.

1.2 ORP initiated the investigation of Montgomery Locks and Dam by their Periodic Inspection Report.[1] The report reviews the construction and the general condition of the locks and dam with attention to specific problem areas. The results of the periodic report warranted further study. A condition survey[2] was conducted by WES for ORP and was accomplished for the dam. This survey determined the concrete quality and location of cracks that could affect the structural integrity of the dam. The present project was initiated to determine if a need exists for a study to consider the rehabilitation or replacement of the locks and dam. If such a need is found, a separate study will be initiated to evaluate the feasibility of rehabilitation or replacement.

Location of Study Area

1.3 Montgomery Locks and Dam are located on the Ohio River in Beaver County, Pennsylvania, 31.7 miles downstream of Pittsburgh.

Purpose and Approach

1.4 The purpose of this report is to present the findings and conclusions drawn from the condition survey and structural investigation

of Montgomery Locks and Dam. This study does not evaluate the foundation, steel gates, bridge work, lock gates, or appurtenant mechanical or electrical facilities which will be considered by ORP when the overall condition of the locks and dam is evaluated.

1.5 This investigation is limited to:

a. A crack survey of the locks.

b. Examination of major cracks under load.

c. Soniscope investigation of several major cracks.

d. Structural property determination of foundation and concrete.

e. Stability analysis of locks and dam monoliths.

f. Stress analysis of selected monoliths.

The evaluation of the structures, as given in this report, is relative to these specific studies; although concrete integrity, concrete deterioration, conditions which may cause immediate failure, existence and extent of structural cracking, and the alignment or settlement of the various structural monoliths were given consideration.

Historical Construction

1.6 A detailed history of the locks and dam features and construction is presented in Reference 1; therefore, only a general description of the locks and dam is given below.

1.7 Montgomery Locks and Dam were constructed under two contracts, from June 1932 to June 1936. They replace single 110- by 600-ft locks and original movable wicket dams No. 4, 5, and 6 which were built in 1898-1908, 1889-1907, and 1892-1904, respectively. The pool extends to Dashields Locks and Dam, river mile 13.3.

1.8 Montgomery Dam is a non-navigable high-lift gated structure. The dam is comprised of a controlled spillway consisting of 10 vertical-lift gated sections, each 100 ft in length having a full vertical travel of 40.25 ft above the sill which is at elevation 667.0. It has an uncontrolled spillway consisting of two fixed weir sections, one 109.5 ft in length adjacent to the abutment and the other 109.25 ft in length

adjacent to the river wall, both with a crest at elevation 680.33. The overall length of the dam between the river wall and the abutment is 1,378.75 ft, including the fixed weir sections and the gated spillway.

1.9 The locks consist of two adjacent parallel lock chambers located along the left bank of the river, a landward 110- by 600-ft main chamber, and a 56- by 360-ft auxiliary chamber.

1.10 The upper guard wall is 403.96 ft long and the upper guide wall is 1080.94 ft long measured from the upstream nose of the middle wall, respectively. The lower guard wall is 180.33 ft long and the lower guide wall is 490.33 ft long measured from the downstream nose of the middle wall, respectively. The upper and lower service gates are of the mitering type and are hydraulically operated. The floors within both chambers are paved.

1.11 The elevation of the top of lock walls, guide, and guard walls is 692.0 ft msl, except at the areas adjacent to the miter gate recesses of the main chamber; here the top of land wall is at elevation 694.0 ft msl.

1.12 Vertical lift from the lower pool, elevation 664.5, to the upper pool, elevation 682.0, is 17.5 ft. Controlling depth in the lower lock approach is governed by the poiree dam foundation which is located 14.6 ft below lower pool.

1.13 The filling and emptying of the lock chambers is accomplished through longitudinal culverts in all three walls.

1.14 Provisions are made for closures of both chambers. These closures cannot be classed as emergency type closures; in the 56-ft chamber a cofferbeam and needle type closures are used and in the 110-ft chamber trestles are raised up out of recesses provided in the bottom of the chamber upstream of the miter gates, and bulkheads are fitted in between each of the trestles. Embedded metal anchorages are provided in the lower guard sill of both chambers for installation of a poiree trestle type dam to permit unwatering of the lock chambers for maintenance and repair work on the lower lock gates. All the necessary materials needed to make the closures are stored at ORP's Warehouse and Boatyard located on Neville Island on the Ohio River at river mile 7.5.

1.15 The above description gives an overall idea of the construction and if more details are desired, they can be found in Reference 1.

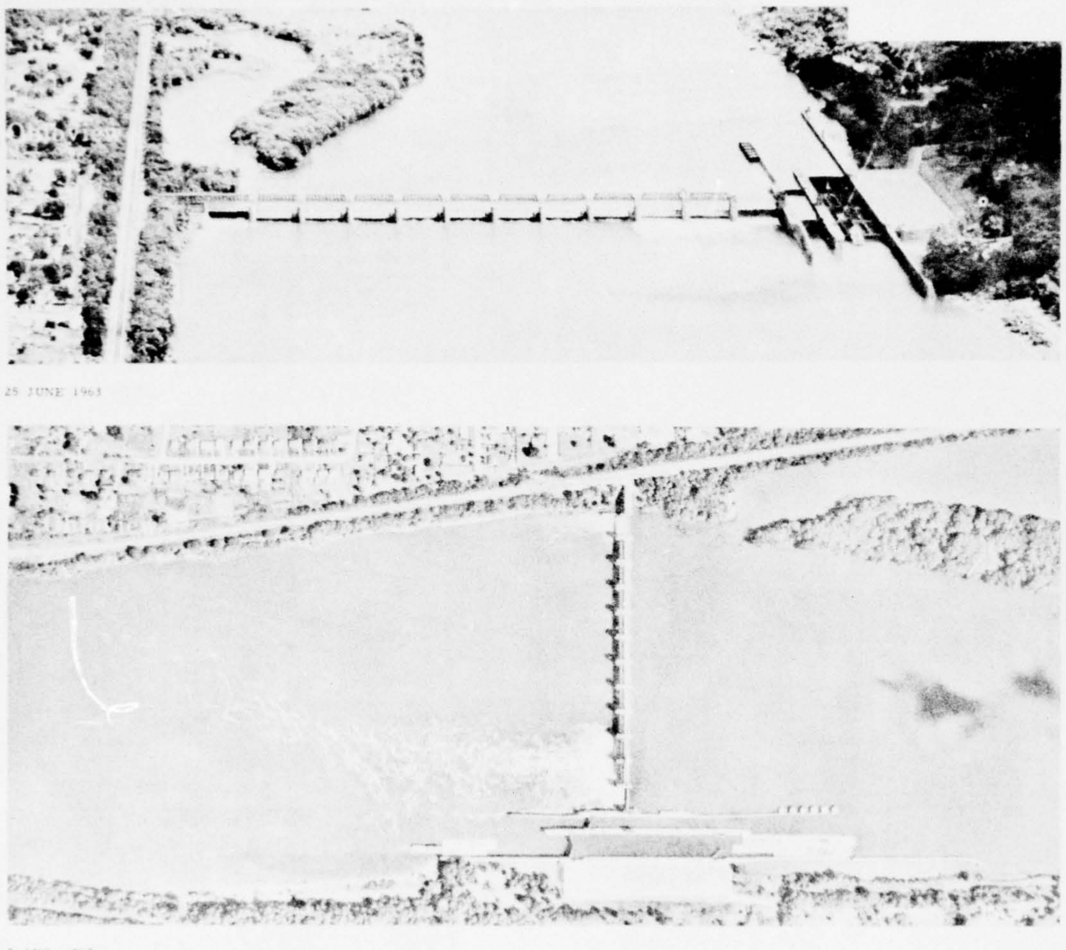


Figure 1.1. Aerial view of Montgomery Locks and Dam.

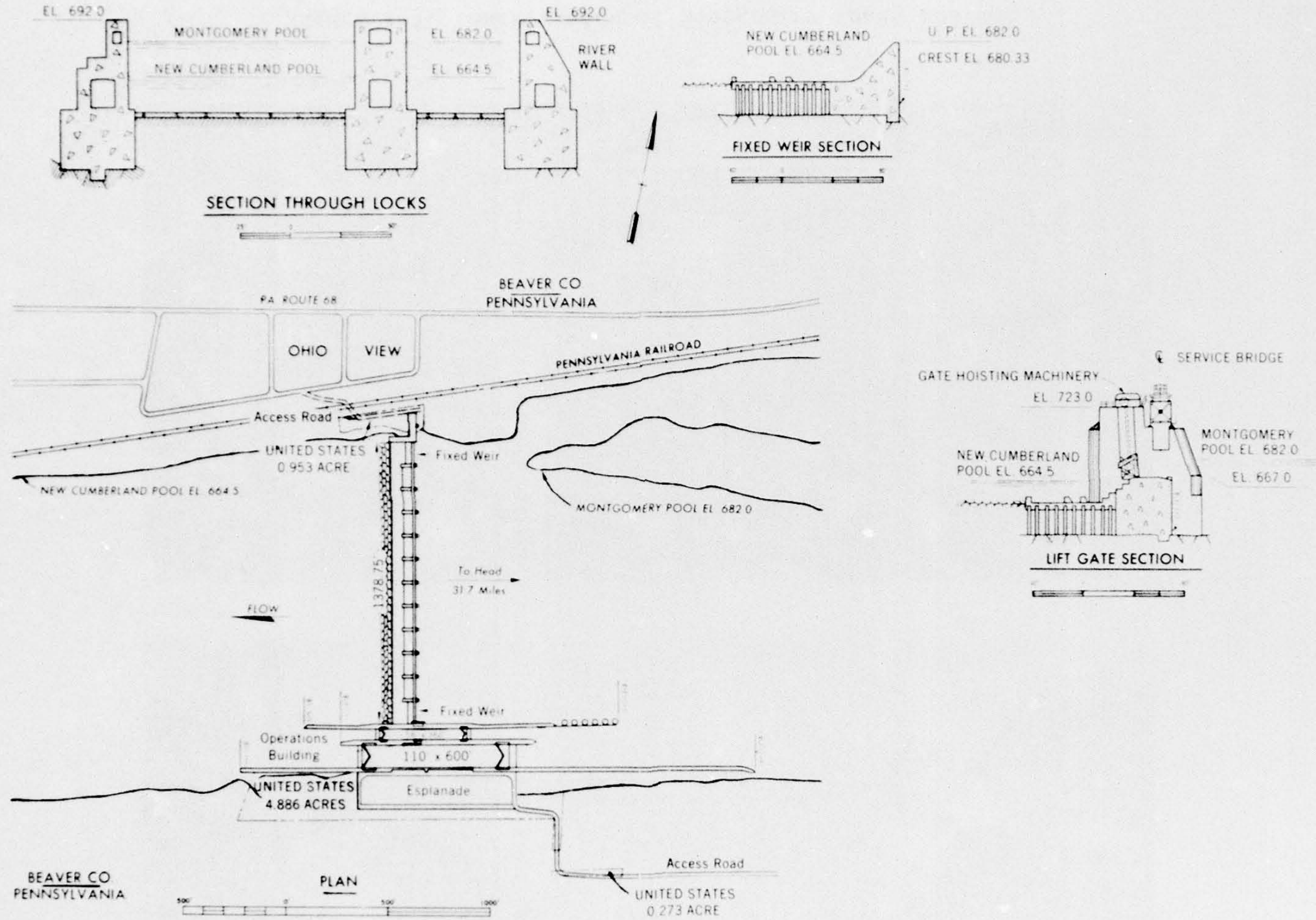


Figure 1.2. Plan and sections of Montgomery Locks and Dam.

SECTION 2: SURFACE CONCRETE DETERIORATION AND CRACKING

Introduction

2.1 An overall view of Montgomery Locks reveals general spalling, leaching and cracking of the concrete surfaces. The surface condition is to the point where accelerated deterioration will occur.

2.2 In certain lock monoliths there are wide cracks (5/32 in.) which are clearly structural and need to be examined. This examination will try and determine the extent and cause of the cracks. These studies will involve:

- a. Logical deductions from physical observations of the structure.
- b. Instrumenting and monitoring major cracks for movement when the locks are exposed to various combinations of moving and static loads under normal operating conditions.
- c. A crack survey (mapping of cracks onto scaled drawings) not only to serve as a record and a means of communication concerning existing cracks but also to establish trends and patterns of cracking.
- d. Soniscope examination of major cracks to determine their extent.
- e. Stress analysis studies.

2.3 In the dam there exists, as it does in the locks, a lot of leaching and cracking. Much of the surface cracking is found in the upper three lifts of the dam piers.

2.4 In the locks and dam, there are groups of parallel cracks and leaching which indicate deterioration due to freezing and thawing and render the areas highly susceptible to water penetration and future freezing and thawing action. The surface cracks will be discussed in detail in the remainder of this chapter.

Results of Monitoring Cracks for Movement

2.5 In the lock walls, the cracks which are of apparent concern are:

a. Numerous transverse and some longitudinal cracks in the guide and guard walls.

b. Major cracking showing in the upstream land wall butterfly valve and bulkhead shafts and significant cracking appearing in the downstream shafts.

c. A predominate crack which extends along the center of the lock chamber middle wall parallel to the lock.

d. Cracking in the upper gate monoliths of both the middle and river walls.

The monolith numbering and stationing is shown in Figure 2.1 It was decided to first monitor the cracks in:

a. The upstream land wall butterfly valve monolith. Cracks and gage locations are shown in Figure 2.2.

b. The upstream middle wall gate monolith. Crack and gage locations are shown in Figure 2.3.

c. The upstream river wall gate monolith. Crack and gage locations are shown in Figure 2.4.

2.6 These major cracks were monitored to see if there was any movement when changes in loadings were imposed upon the locks. Combinations of loading were induced by:

a. Changing pool levels.

b. Opening and closing gates.

c. Opening butterfly valves allowing water to enter the lock.

The checking for crack movement under combinations of water, gate and butterfly valve loadings helps in considering the cause and importance of the cracks.

2.7 The crack in the upstream land wall butterfly-valve monolith was monitored from the top of the monolith and also from the vertical face within the butterfly-valve shaft. The other two cracks were monitored only from the top of the monolith.

2.8 The measurements determining crack movement under water, gate, and valve loadings were accomplished with a Dimic Gage having a 0.00001 in. accuracy. There was no detectable crack movement under any combination of loading. The relative movement across the cracks were monitored by the principles given in ETL 1110-2-118 except a more precise instrumentation was used which would detect movement of

0.00001 in. No detectable crack movement implies that the gate and valve loadings are not significantly affecting the existing cracks. The stress analysis of loaded monoliths is another means by which the cause of the cracking can be considered. The stress analysis implies the causes of certain cracks; this discussion is given in Section 7.

Crack Survey

2.9 An important indication of the condition of the locks and dam is the degree, extent, and cause of surface cracking. The surface cracks on Montgomery Locks were mapped onto scaled drawings and are presented in Figures 2.5 through 2.13.

2.10 The cracking in the locks will be discussed and correlated starting with the upstream land side monoliths except when general trends are observed; they will then be discussed in relation to the other portions of the locks which establish the trend.

2.11 As the cracks were studied, it became apparent that a distinction between their degree and importance needed to be delineated. The delineation as given in Table 2.1 was used to categorize the cracks by width and extent. This in effect classified the cracks according to importance as they appeared from the surface observations.

2.12 The upper guide wall extension is in good condition and only has surface cracking in the upstream end monolith. The original upper guide wall has significant transverse cracking much of which extends to a considerable depth as can be seen on either side of the monoliths. A trend of cracking becomes apparent in the guide and guard walls. The extent of cracking in guide and guard walls may be rated from the most to the least, both in number and degree, as follows: upper guide wall, lower guide wall, upper guard wall, and lower guard wall. There is a lot of cracking in the lower guard wall but the cracks are smaller in width and less in extent. The main cracks in these walls are located close to the middle half of the monolith and extend from the water line on river side to the water or fill line on land side. This total trend of cracking leads to the hypothesis that

their origin is probably due to barge impact loads. The frequency and the intensity of barge impact are both important in affecting the monoliths. This leads to the conclusion that the smaller lock which carries light traffic and is next to the guard walls should be subjected to lighter impact loads than the guide walls. Also, seemingly a barge headed downstream would create a larger impact than one headed upstream due to the greater momentum caused by the current. This logic makes the ranking of cracking from most to less severe in the upper guide wall, lower guidewall, upper guard wall and the lower guard wall. This is the case (except the cracking in the lower guide and upper guard wall is about the same) and fits the trend that the origin of the cracks is barge impact.

2.13 Now, considering the cracking in the land wall, the most severe cracking is in the upper portion of the wall. This trend is continued across the three walls in that the most severe cracking is in their upper ends. Again a barge headed downstream would create a larger impact than one headed upstream; therefore, the impact force would tend to be larger and cracking more significant on the upstream portion of the lock walls. This could well be the reason the most significant cracks exist in the upstream portion of the lock walls at Montgomery Locks and Dam.

2.14 In the stress analysis given in Section 6, it is seen that the barge impact can cause the center line cracking parallel to the lock due to excessive tensile stress and cracking in culvert walls allowing stress flow upward through the center of the monolith. The impact stresses were relieved by transverse cracking in the guide and guard walls but due to stress concentrations at changes in geometry of culverts and block outs it is relieved by longitudinal cracking in the lock chamber monoliths.

2.15 Center-line cracks in the lock walls exist mostly in their upstream half; except, the land wall also has significant center-line cracks in its downstream butterfly-valve and bulkhead shafts. In the shafts where center-line cracking is significant the cracks can be seen to extend to and below the water line at el 664.5. The trend is that cracks seen in the monolith surface which extend into the pipe-gallery

ceiling and vertical walls can not be seen in the floor. An exception to this is seen in monolith M-8 of the middle wall where a center-line crack exists in the floor of the pipe gallery. These center-line cracks could be related to impact loads causing high stress concentrations to develop around openings, storage bays, pipe galleries, and conduits.

2.16 The cracking in the structural slab over the storage area in the upper land wall is longitudinal down its center, extends down the face of the end walls and continues into the bulkhead recess. This cracking cannot be seen in the floor of the storage pits.

2.17 The cracks seen in the tunnels of the land wall monoliths are shown in Figures 2.7 and 2.8. Some transverse cracking in monolith L-17 shows on the surface and also in the pipe gallery. The center-line cracking in the land wall butterfly-valve monolith L-19 shows up in the pipe gallery ceiling and walls and in the butterfly-valve recess. This cracking extends back into monolith L-18. One transverse crack can be seen across the floor which indicates there could be some depth to this cracking. This is one of the cracks studied by pulse velocity work and will be discussed later in this section. The cracking in the end of the butterfly-valve monolith is not surprising because of the large cut out which leaves only a small longitudinal thickness of concrete to carry any stresses induced perpendicular to the center line. In monolith L-29 and L-30 the cracks in the top surface can be correlated with those in the ceiling and walls of the pipe gallery and in the faces of the bulkhead shaft.

2.18 The downstream bulkhead has cracking which extends to some depth as can be seen in the bulkhead recess. There is cracking in the rest of the land wall but as mentioned earlier it is less severe than that in the upper half of this wall.

2.19 The lower guide wall has significant cracking as can be seen from Figure 2.6 but it is less severe than that in the upper guide wall. The main cracking is in its lower end where it experiences the worse impact loadings.

2.20 The predominate cracking in the middle wall is along its center parallel to the lock. In Section 6 the reason for this cracking

is hypothesized as being caused by barge impact. This idea is substantiated by the fact that the longitudinal crack is not predominate in the gate bay monoliths. The monoliths could still get impact from the river-side lock but the river lock handles lighter traffic. These longitudinal cracks show in the ceiling but not in the floor except in monolith M-8. Transverse cracking occurs in most upper guard wall monoliths. These cracks are usually located close to the middle half of the monolith, with the more significant cracks extending down to the water surface and ranging in width up to 5/32 of an inch.

2.21 A transverse crack similar to those on the upper guide wall exists near the midpoint of monolith L-9. This crack is less than 5/32 of an inch in width and extends down to the water surface on both sides.

2.22 Significant cracks are in the upper end of the river wall (monoliths R-10, R-11, R-12, and R-13). Some of these cracks reach widths in excess of 5/32 in. and extend down to the water surface in the shafts.

2.23 In summary the significant transverse cracks in the lock walls correlate with those in the ceiling and walls of the pipe galleries. The longitudinal cracks correlate with cracks in the ceilings of the galleries but are not seen (except in M-8) in the floors. There is strong evidence that much of the cracking is caused by barge impact. Barge impact would not necessarily have to cause cracking in the center of the gallery floors but could be relieved by cracking the ceilings mainly in the center because of the comparatively thin slab of concrete above the gallery. Many of these cracks discussed above appear in the pipe gallery and in the land and river faces of the monoliths which will allow freezing and thawing action to deteriorate the monoliths to a considerable depth. The cracks should be sealed from the weather and some of the larger ones monitored to see if they become worse; barge impact could cause the cracks to progressively become worse and deteriorate the monoliths.

2.24 A pulse velocity study was made to determine the depth of the cracks beyond that which is apparent from the surface and from observations within tunnels and other blockouts within monoliths.

Pulse Velocity Investigation

Background

2.25 The ultrasonic pulse velocity technique is an accepted technique for nondestructively studying the integrity of concrete structures. It can give useful information regarding the quality of the concrete and in many instances, the severity of cracking.

2.26 In surveying a structure using this technique, it is desirable to send a pulse through a wall, or walls, in as many areas as accessibility will permit. At selected points along a wall the transmitter and receiver tranducers are placed opposite each other on either side of the wall and a pulse imparted to the concrete. If the concrete is of good quality, the signal will be picked up after passing through the wall. A mass of concrete of excellent quality will have high pulse-velocity magnitudes while poor quality will yield low velocities. Cracks will significantly reduce both the velocity and the strength of the signal. Experience in ultrasonic testing indicates that the relation between velocity and quality of concrete of normal density is approximately as shown in the following tabulation. It should be noted, however, that these values are only typical and cannot be expected to apply in all instances.

Pulse Velocity, fps	Condition
Above 15,000	Excellent
12,000 - 15,000	Generally good
10,000 - 12,000	Questionable
7,000 - 10,000	Generally poor
Below 7,000	Very poor

2.27 Ultrasonic pulse-velocity measurements were made in the lock land wall, middle wall, and river wall at Montgomery Locks and Dam on the Ohio River, Pittsburgh District. The primary objective of this study was to attempt to determine the depth of several visible surface cracks at specific locations on the lock walls. One borehole was drilled on each

lock wall in the vicinity of the cracks of interest. These boreholes were drilled through the wall to the base of the structure.

2.28 Most of the measurements were made using borehole transducers in conjunction with the soniscope. These transducers require water as a couplant to the structure; therefore, the transducers are used only in situations where areas of interest are below water level. One transducer was inserted in the water-filled borehole and the other along the side of the lock wall below water level.

Land Wall Measurements

2.29 The first set of measurements on the land wall was from the borehole which was 20.25 ft from the lock wall in monolith No. 19 at station 3 + 39.25A. Measurements were begun with both transducers located 15 ft below the deck and continued at 5-ft intervals to a depth of 40 ft. Both gages were then lowered to the bottom of the lock pool, 42 ft below deck, and a velocity measurement was made. The pool-side transducer was held at -42 ft and the borehole transducer was lowered at 5-ft intervals to -75 ft, and velocity measurements were made at each location. An additional measurement was made with the borehole transducer at -78 ft.

2.30 The pulse velocity range was from 13,720 to 14,411 fps with an average velocity of 14,110 fps. Three data points were not included (between 25 and 40 ft) because the path was through a 13-ft gallery filled with water. The average velocity for these three points was 6060 fps. These and other velocity data are shown in Table 2.2.

2.31 The second set of measurements on the land wall was from the borehole which was 25.3 ft from the lock chamber in monolith No. 18 at station 3 + 13.95A. Measurements were begun with both transducers located 15 ft below the deck and continued at 5-ft intervals to a depth of 40 ft. Both gages were again lowered to 42 ft and measurements were continued holding the flooding chamber transducer at 42 ft and lowering the borehole transducer to a depth of 78 ft in increments of 5 ft. The pulse velocities ranged from 13,177 fps to 14,560 fps with an average velocity of 14,054 fps.

2.32 The preceding two sets of measurements did not show any strong evidence of excessive cracking below the water level. However, small- to medium-size cracks which are filled with water would not affect the velocity

magnitude significantly unless they are numerous. The velocities obtained between the borehole and the lock chamber were, in some cases, sufficiently low to allow a classification of "suspect area."

2.33 A third set of measurements was taken from the top surface of the land wall, 14.7 ft to the lock wall, across a visible crack in the butterfly valve monolith. The transducer on the top surface was held in place while the wall transducer was moved downward in 7 steps from 1 ft to 42 ft from the surface. Only two successful measurements were obtained. At 15 ft down the lock wall, a velocity of 7368 fps was obtained and at 25 ft down, a velocity of 7702 fps was obtained. It can then be concluded that this crack most likely extends to, or nearly to, a depth of 42 ft.

2.34 Measurements were also made from the borehole to the lock wall face at a depth of 25 ft below the deck. The lock wall transducer was moved at 6-ft intervals upstream along the lock walls. Velocity measurements were made at every 6-ft interval. The first 3 measurements gave velocities of 13,846 to 14,585 fps, which would show no significant cracking. The additional measurements were across monoliths and gave unacceptable data.

2.35 Measurements were taken with a surface transmitter placed 8 ft below the top deck on the vertical face of the butterfly valve wall at station 3 + 56.0A and with the borehole receiving transducer located at 8 depths in the lock pool ranging from 11.6 ft to 42 ft. No data were obtained at depths 35 and 42 ft. The data for the remaining six locations ranged from 11,817 to 13,112 fps with an average velocity of 12,449 fps. These values indicate that the crack does extend to a depth of at least 42 ft.

2.36 The final ultrasonic pulse velocity measurements on the land wall were taken across the surface from the vicinity of the borehole across several cracks to the vertical wall at station 3 + 21.2A. The transmitter was positioned 1.25 ft below the monolith surface on the vertical wall and the receiver on the monolith surface across from the transmitter. The pulse velocities for the two positions were 9,885 and 9,224 fps, confirming that the cracks are more than just surface cracks.

Middle Lock Wall Measurements

2.37 Pulse-velocity tests were conducted from the borehole which is 14.8 ft from the face of the large lock chamber at station 3 + 00A. Measurements were begun with both transducers located 15 ft below the deck and continued at 5-ft intervals to a depth of 42 ft. The pool-side transducer was held at -42 ft and the borehole transducer was lowered at 5-ft intervals to -75 ft, and velocity measurements were made at each location.

2.38 The pulse-velocity range was from 13,307 to 16,426 fps with an average velocity of 14,587 fps. Four data points were not included (between 30 and 42 ft) because the path was through a gallery filled with water. The average for these four locations was 6328 fps. The magnitude of the pulse-velocity signals were of sufficient magnitude to indicate that there is no severe cracking below water level (12 ft below deck) at this particular station.

2.39 Pulse-velocity measurements were made from the borehole to the wall of the flooding chamber a distance of 11.5 ft across a visible surface crack. Measurements beginning at a point 12 ft below the deck to a depth of 25 ft showed no severe cracking. The velocities ranged from 14,650 fps to 15,092 fps. Measurements were inconclusive from the bottom of the gallery to the bottom of the borehole because of insufficient signal strength to resolve the first arrival.

2.40 Pulse-velocity measurements were made from the borehole to the wall of the small lock through a distance of 8.6 ft. Measurements were begun with both transducers located 12 ft below the deck and continued at 5-ft intervals to a depth of 20 ft (bottom of small lock). The pool-side transducer was held at -20 ft and the borehole transducer was lowered at 5-ft intervals to a depth of 60 ft, and velocity measurements were attempted at each location. To a depth of 25 ft, the velocities ranged from 14,751 fps to 15,237 fps, indicating no severe cracks. Measurements below 30 ft were not successful, indicating possible cracking or lift-joint separations.

2.41 Measurements were made at two locations on the middle lock wall with the borehole transducer located on the opposite sides of the wall. Measurements were made from a depth of 12 ft to 42 ft. The path length for the ultrasonic pulse was 24 ft. There were visible surface cracks at

these locations, but apparently did not extend to a depth of 12 ft because velocities which ranged from 14,414 fps to 15,738 fps did not show significant cracking. At one of the locations, velocity measurements were made close to the top surface of the wall. The signal strengths were very poor, and the velocities were 11,137 fps. These data indicated that the cracks did have a depth of at least 1-1/2 ft.

River Wall Measurements

2.42 Velocity tests were made from a borehole at station 2 + 32.5, monolith No. 12, to the small lock walls at increments similar to previous lock walls. The lock wall transducer was lowered to 24 ft. Because of interference of the lower gallery, measurements could not be made at borehole depths greater than 30 ft. The velocities ranged from 13,616 fps to 13,969 fps. These data do not indicate severe cracking.

2.43 Similar measurements were made with the lock wall transducer moved 13.9 ft downstream. The pulse velocities ranged from 12,926 fps to 14,350 fps. These data, again, do not indicate severe cracking; however, the signal strengths were sufficiently low so as to make this a suspect area.

2.44 Velocity measurements were taken between the borehole and the river wall at depths of 12 ft, 20 ft, and 30 ft with a path length of 2.08 ft. The velocities ranged from 13,506 fps to 14,751 fps, indicating no severe cracking.

Conclusions

2.45 The ultrasonic pulse-velocity tests do indicate in some instances cracking down into the structure. Pulse velocity tests through saturated concrete below water level do not always show cracks because the water fills the space. In some instances the signal strength or transmission time will not be significantly affected by the water-filled cracks, although if the cracks are large or numerous, the pulse velocity will usually be reduced significantly.

2.46 The main concerns for the large cracks at Montgomery Lock and Dam (except for the cracks along the middle lock wall) should be satisfied by instrumenting and keeping a systematic weekly record of the surface crack widths. If the cracks do not increase in width, there is no problem; if

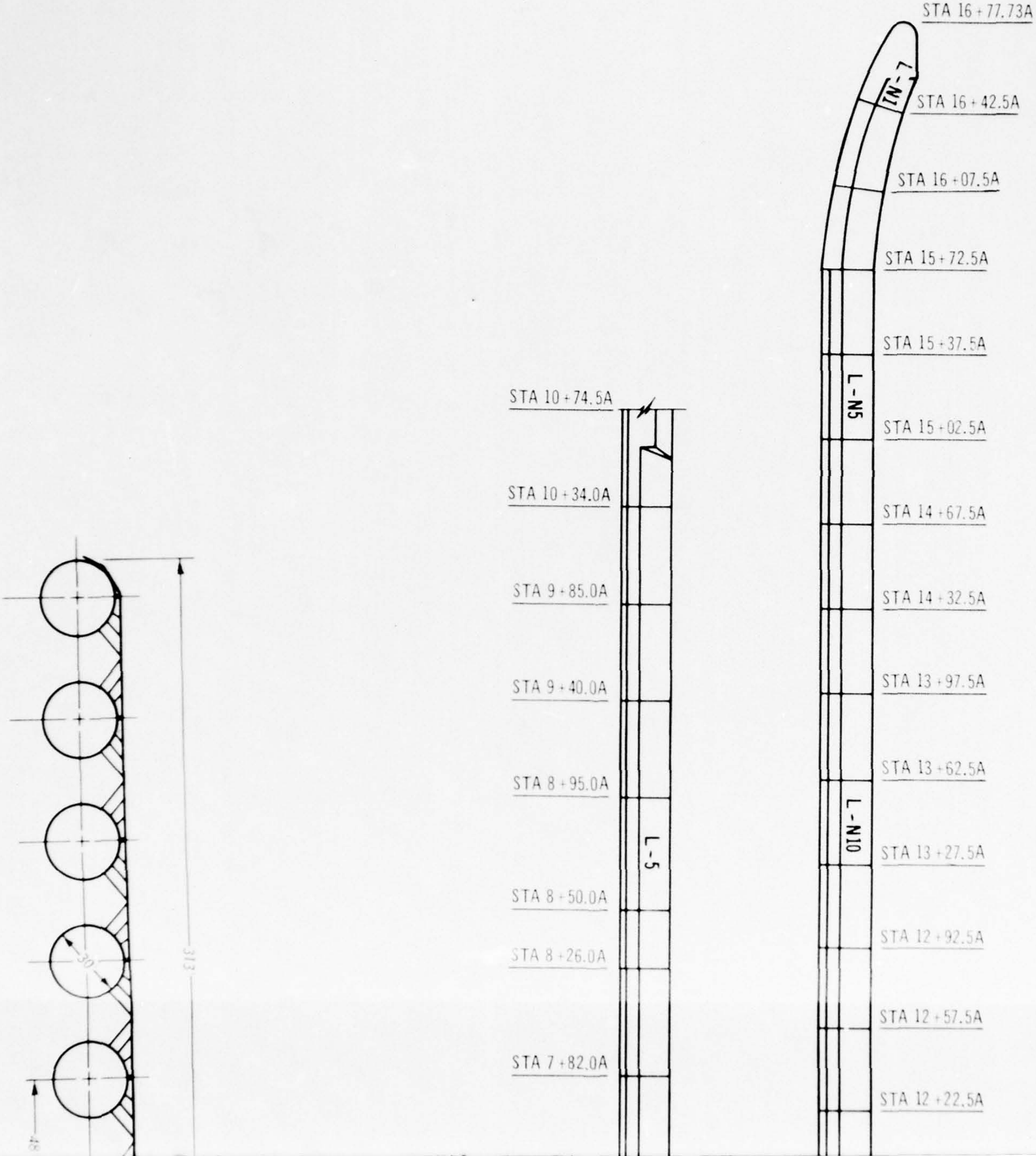
they do, further study is necessary at that time. The crack down the middle lock wall should also be instrumented and weekly records kept of the surface crack width. The lower culvert wall should be checked for cracking and the hypothesis that barge impact caused this crack investigated.

Surface Condition

2.47 The land-, middle-, and river-wall monoliths are cracked, spalled, and in general show signs of progressive deterioration due to weathering. The concrete is nonair entrained which will allow progressive damage due to freezing and thawing. Many open cracks allow water penetration which results in accelerated deterioration due to freezing and thawing.

2.48 The surface condition and the extent of cracking suggests that some action must be taken. As a minimum, the concrete surfaces must be rehabilitated to check accelerated deterioration. It is not possible to recommend the most feasible action without a feasibility study considering the total lock and dam operational and maintenance costs over an extended period of time (30 to 50 years).

2.49 The next step in the evaluation of Montgomery Locks and Dam is to make a feasibility study of rehabilitation or replacement.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L-5
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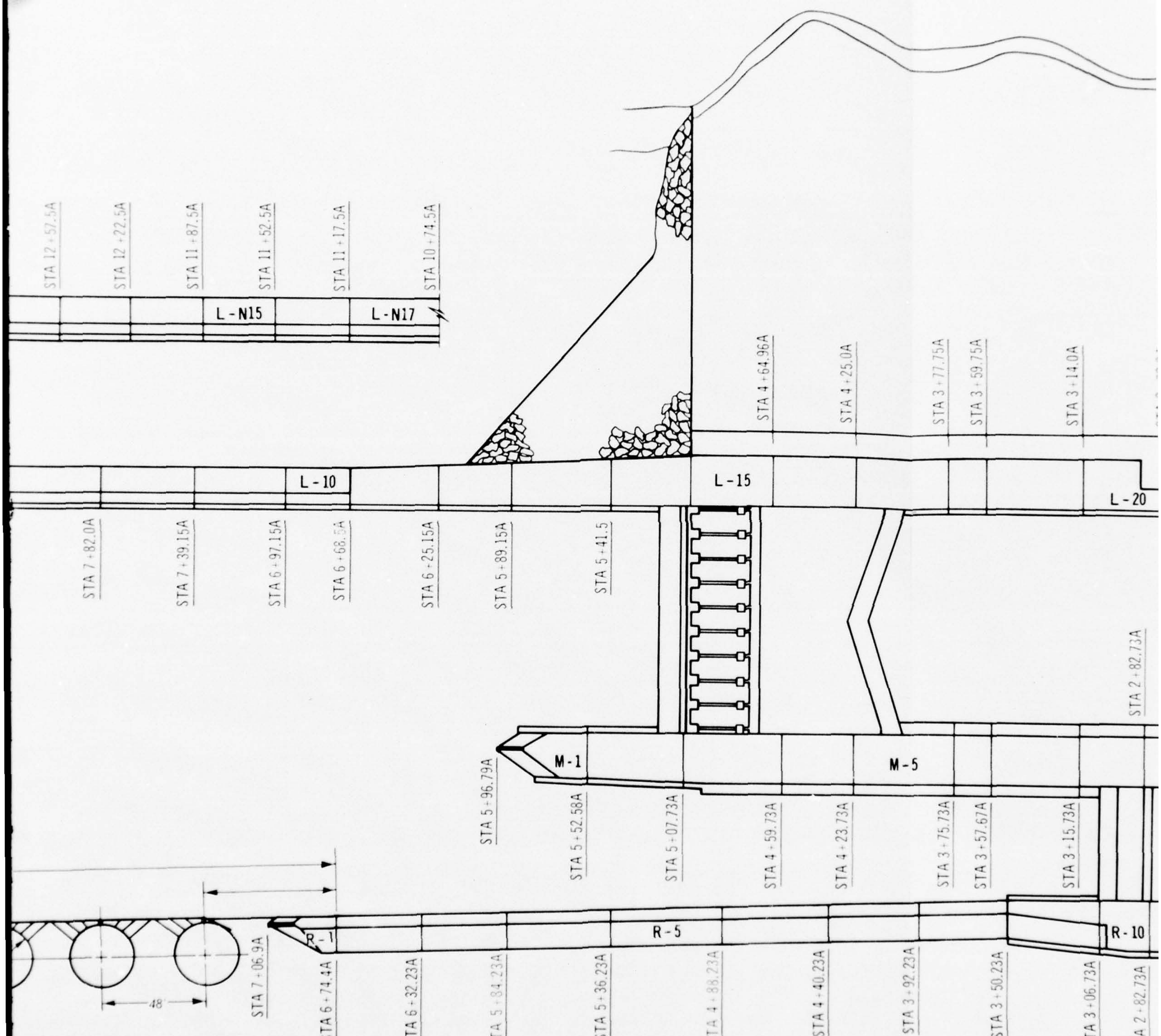
313
30
48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L-20
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M-5
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R-1
R-5
R-10
48'
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STA 6+74.4A
STA 6+32.23A
STA 5+84.23A
STA 5+36.23A
STA 4+88.23A
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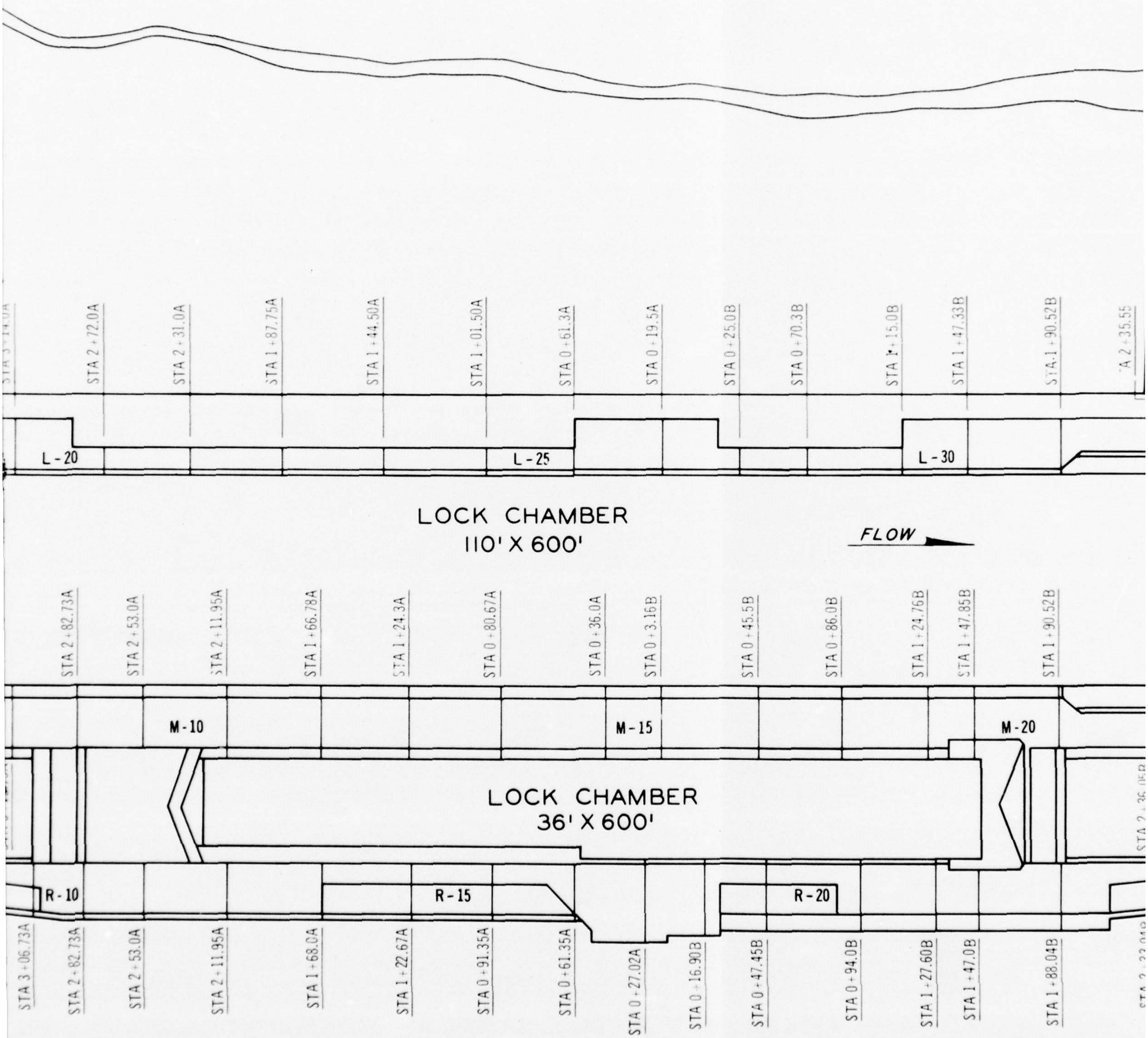
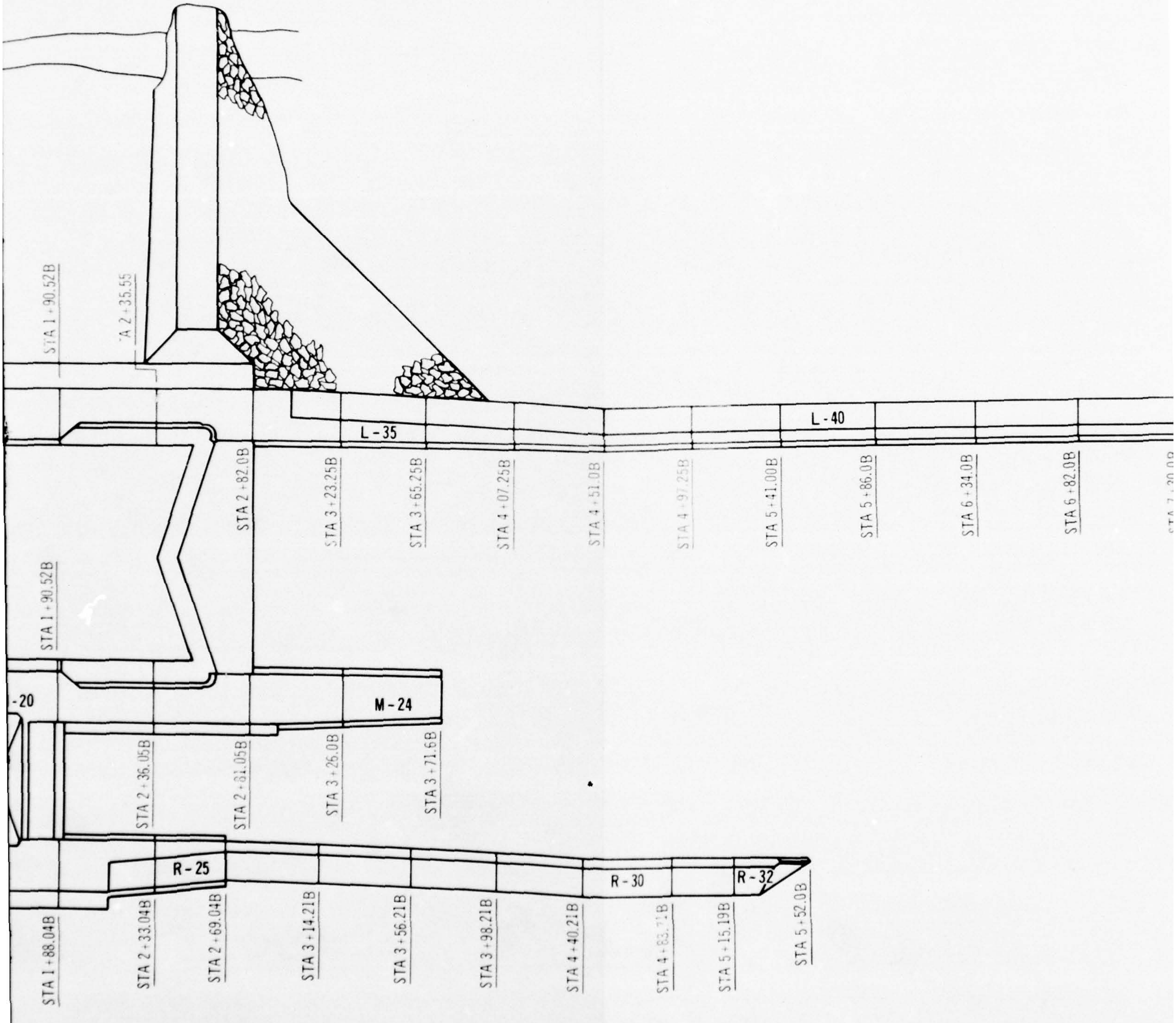
STA 3+50.23A
STA 3+06.73A
STA 2+82.73A

Figure 2.1

2



ure 2.1 Montgomery Lock - monolith and station numbering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STA 6+82.0B
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-20
M-24
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STA 2+81.05B
STA 3+26.0B
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R-25
R-30
R-32
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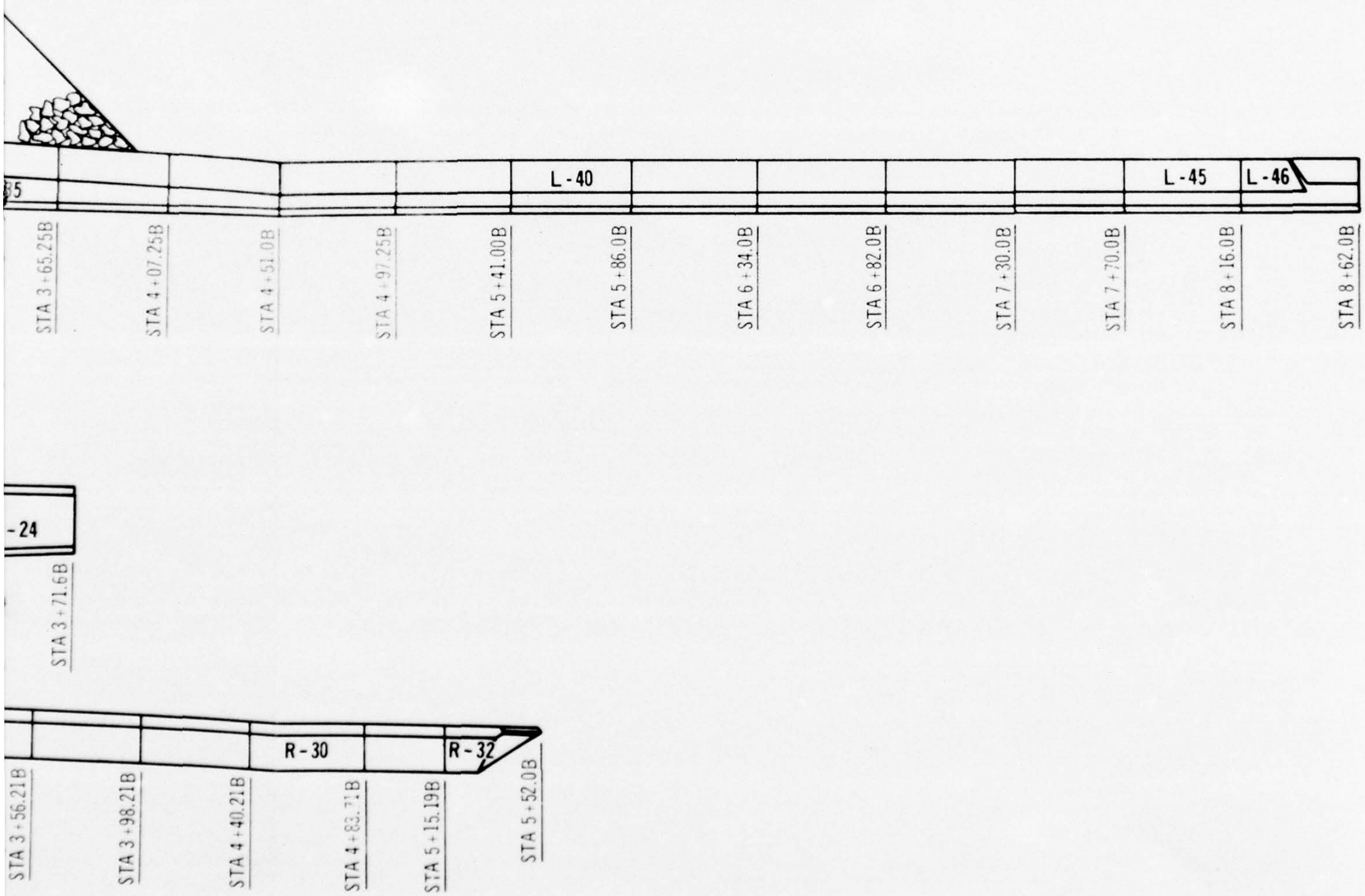
4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STA 8+62.0B
-24
STA 3+71.6B
R-30
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5

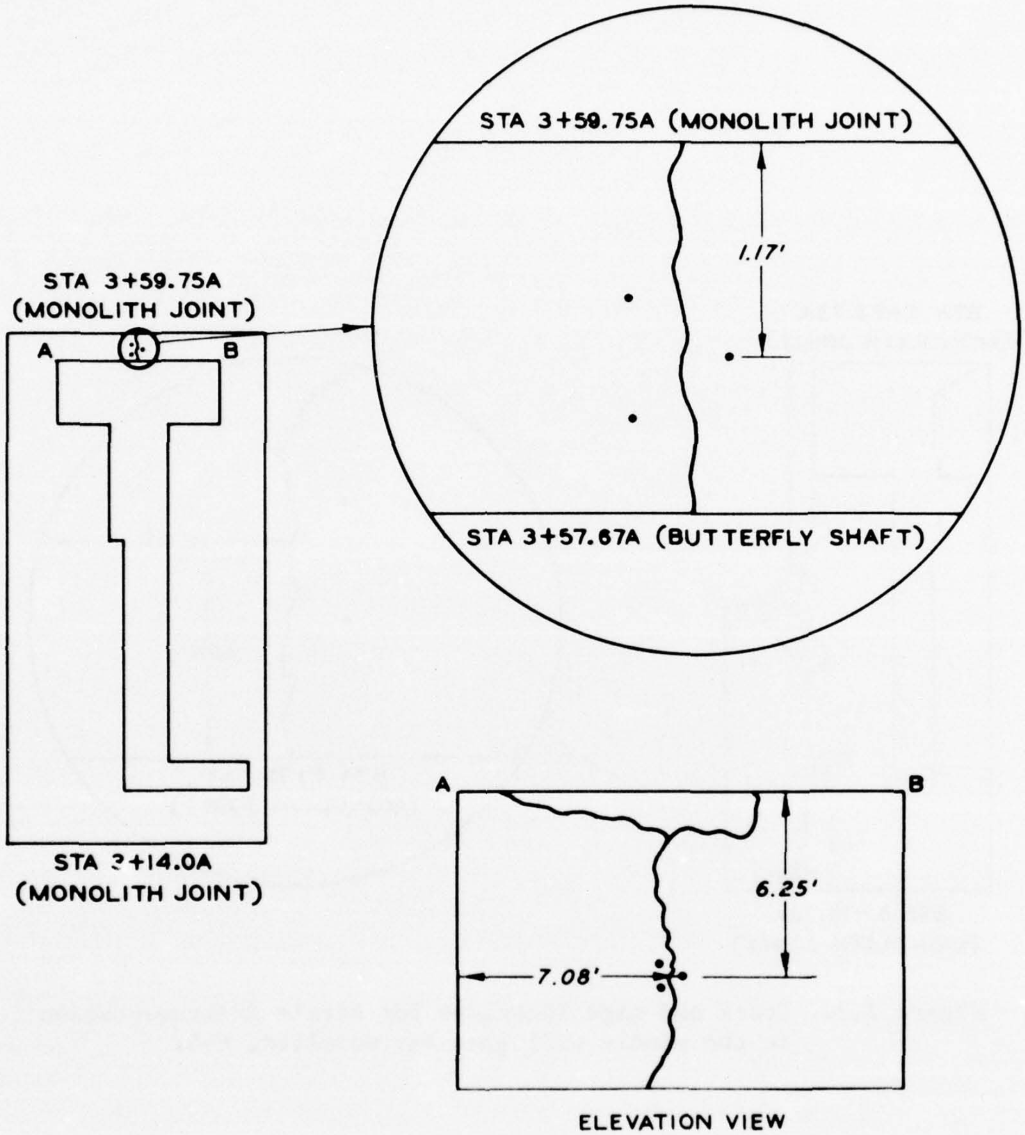


Figure 2.2. Crack and gage locations for strain instrumentation in the land wall butterfly valve monolith, L-19.

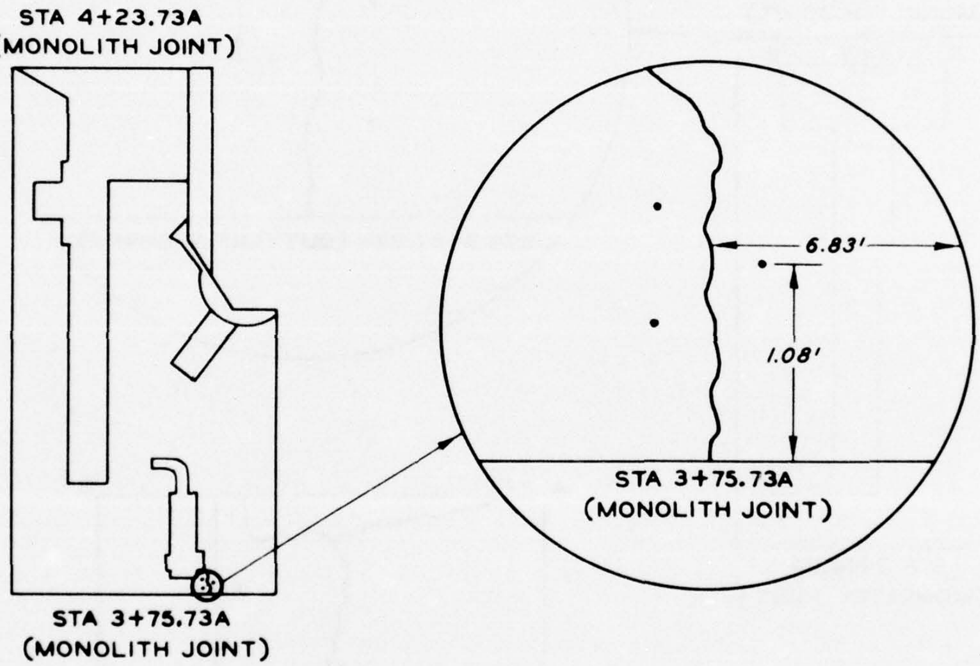


Figure 2.3. Crack and gage locations for strain instrumentation in the middle wall gate bay monolith, M-5.

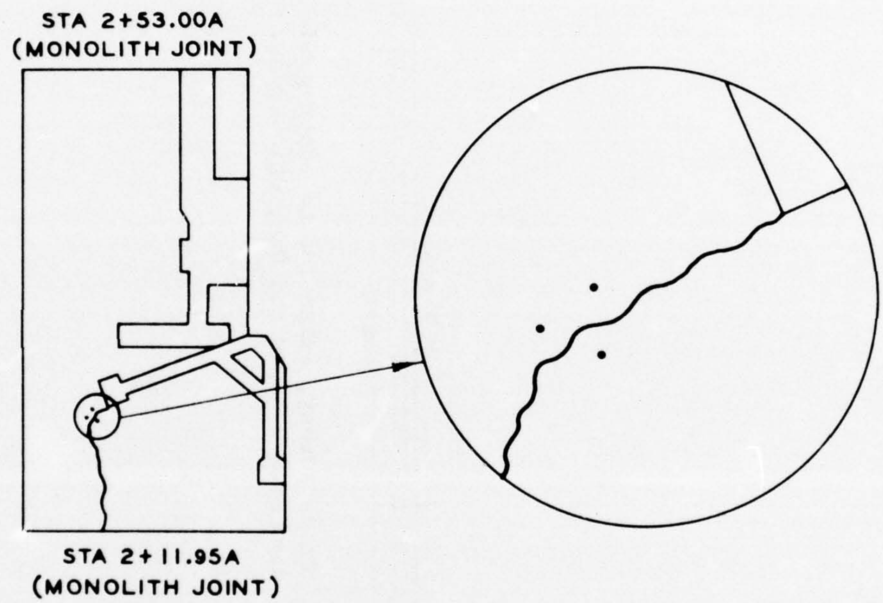


Figure 2.4. Crack and gage locations for strain instrumentation in the river wall gate bay monolith, R-12.

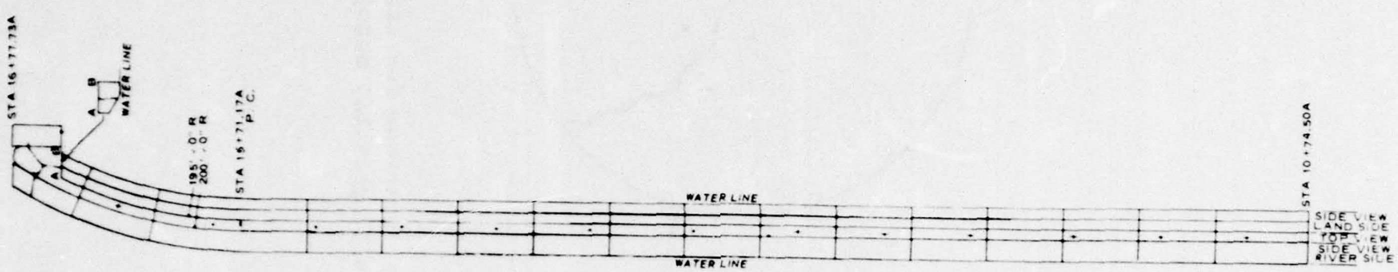


Figure 2.5. Crack survey, extension to upper guide wall, Montgomery Locks and Dam, Ohio River.

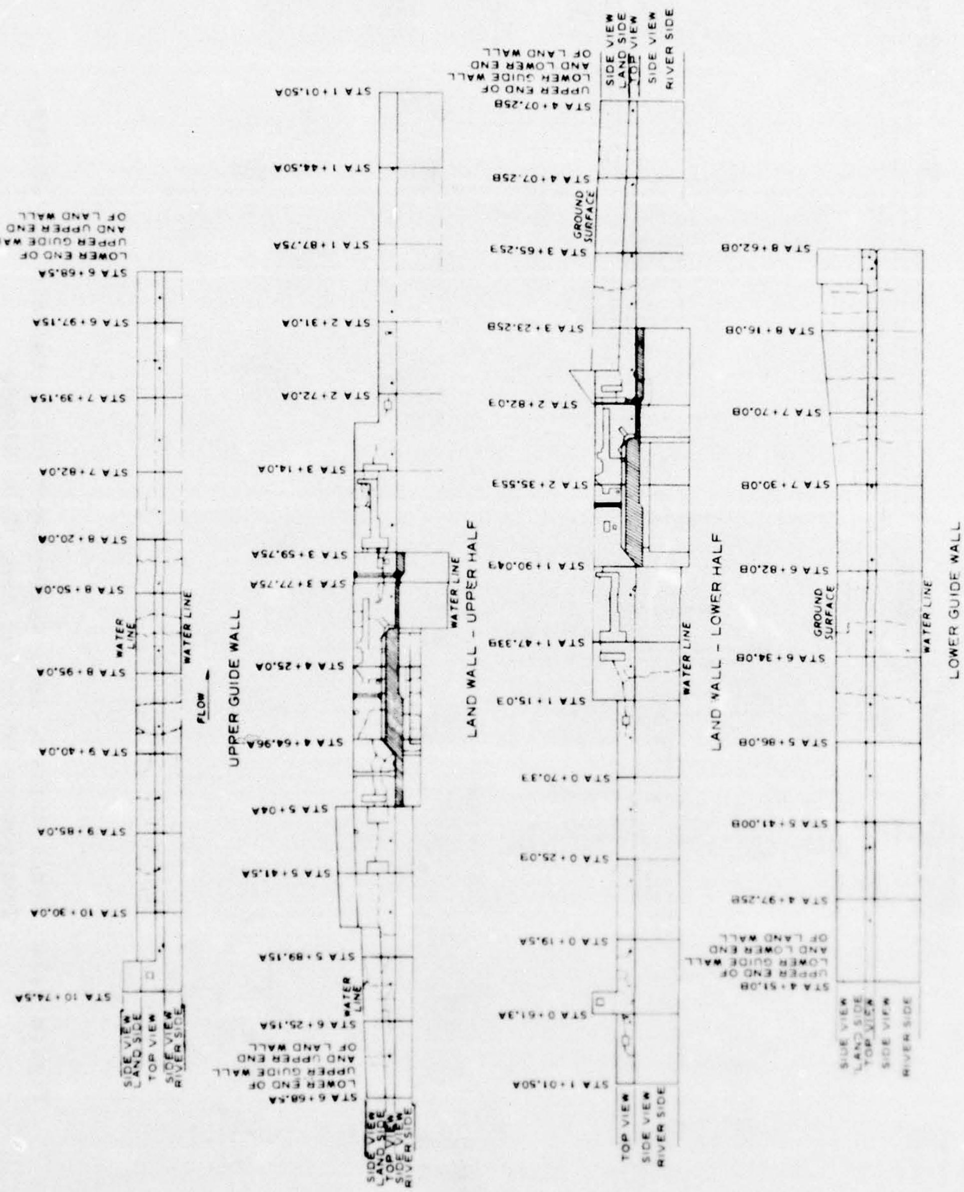


Figure 2.6. Crack survey, land and guide walls, Montgomery Locks and Dam, Ohio River.

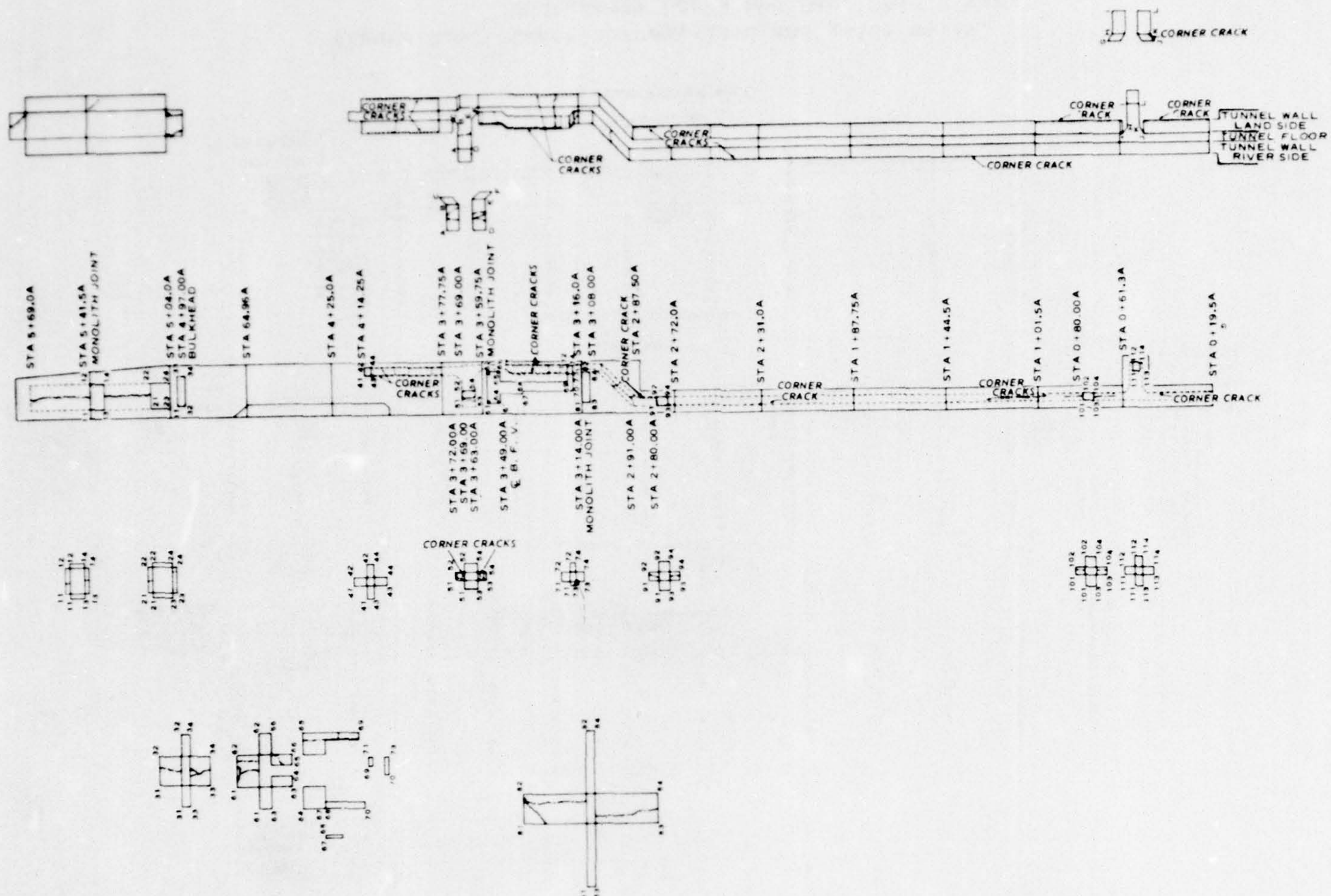


Figure 2.7. Crack survey, land wall--upper half, tunnels and shafts, Montgomery Locks and Dam, Ohio River.

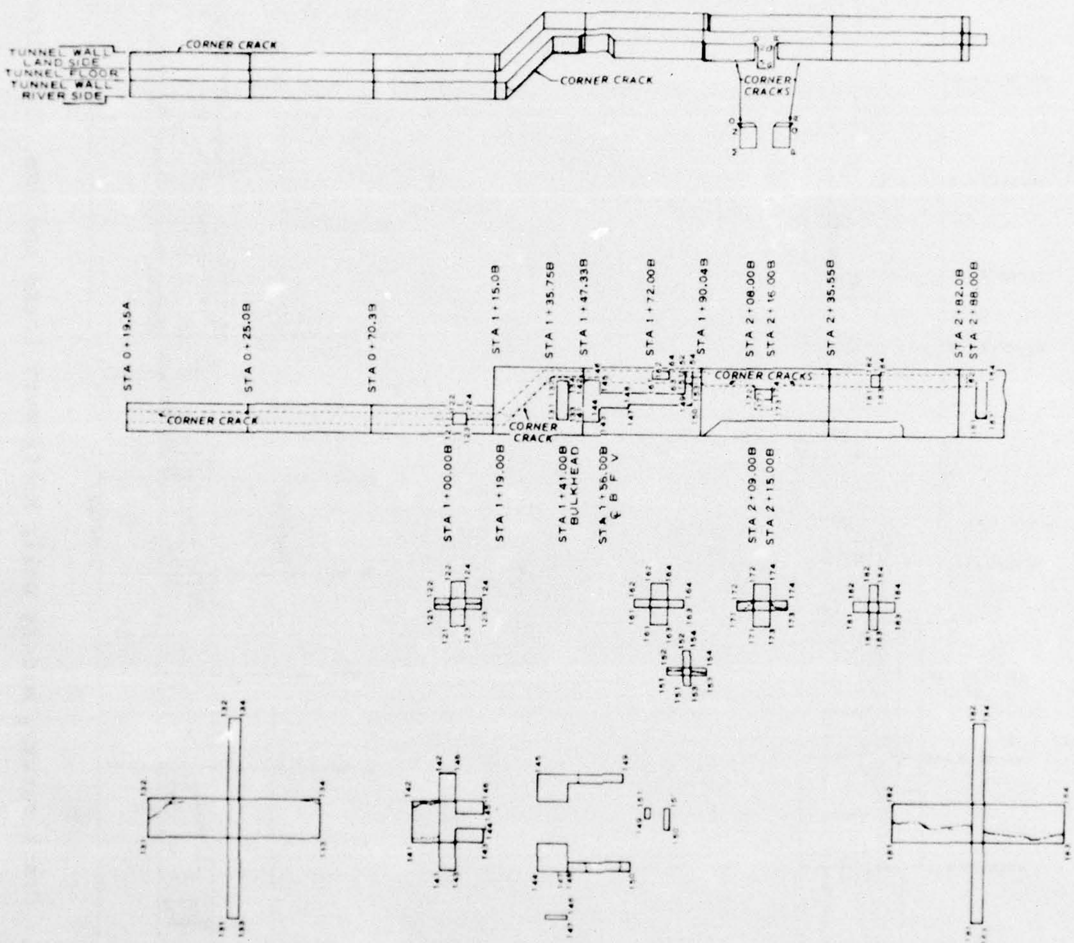


Figure 2.8. Crack survey, land wall--lower half, tunnels and shafts, Montgomery Locks and Dam, Ohio River.

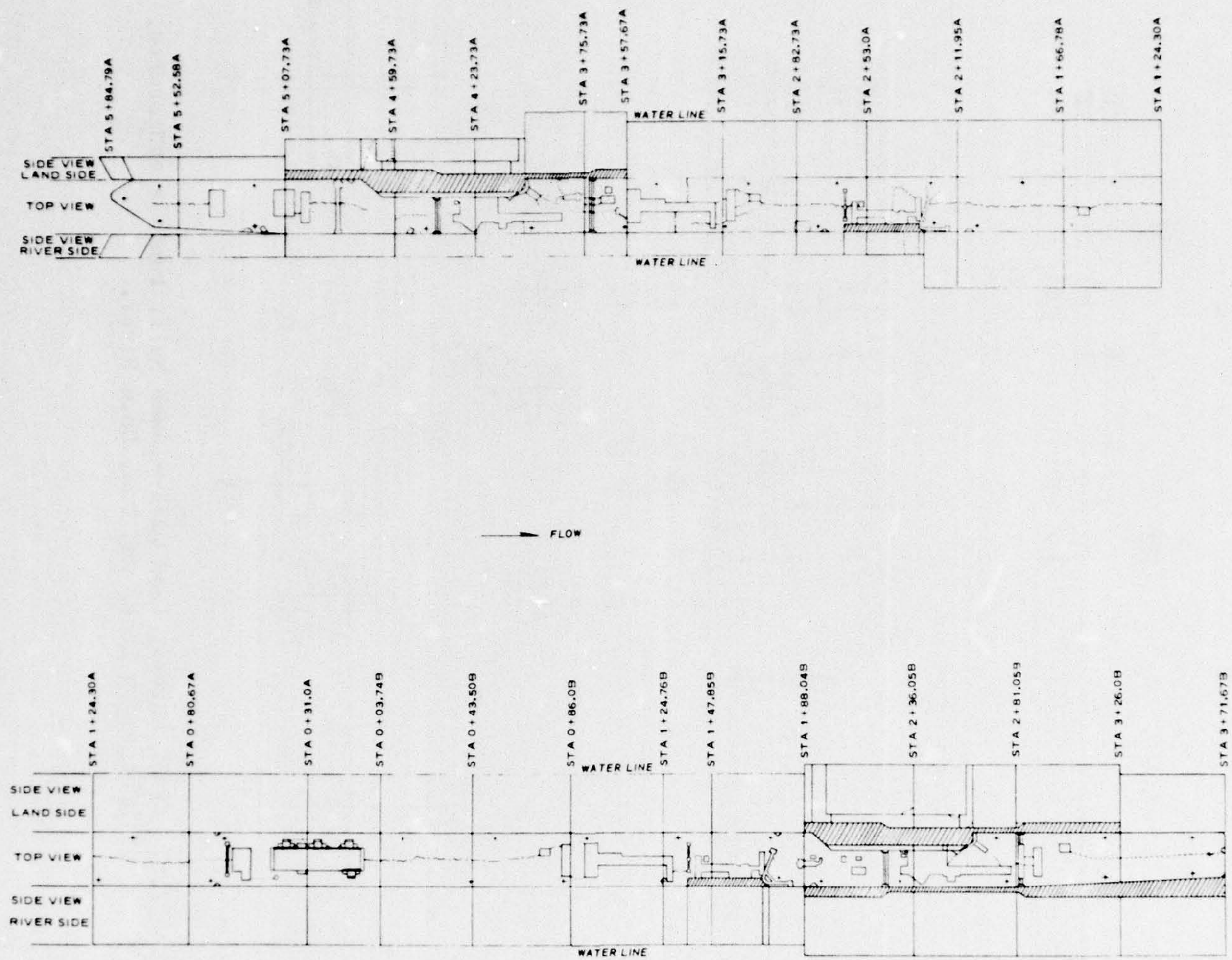


Figure 2.9. Crack survey, middle wall, Montgomery Locks and Dam, Ohio River.

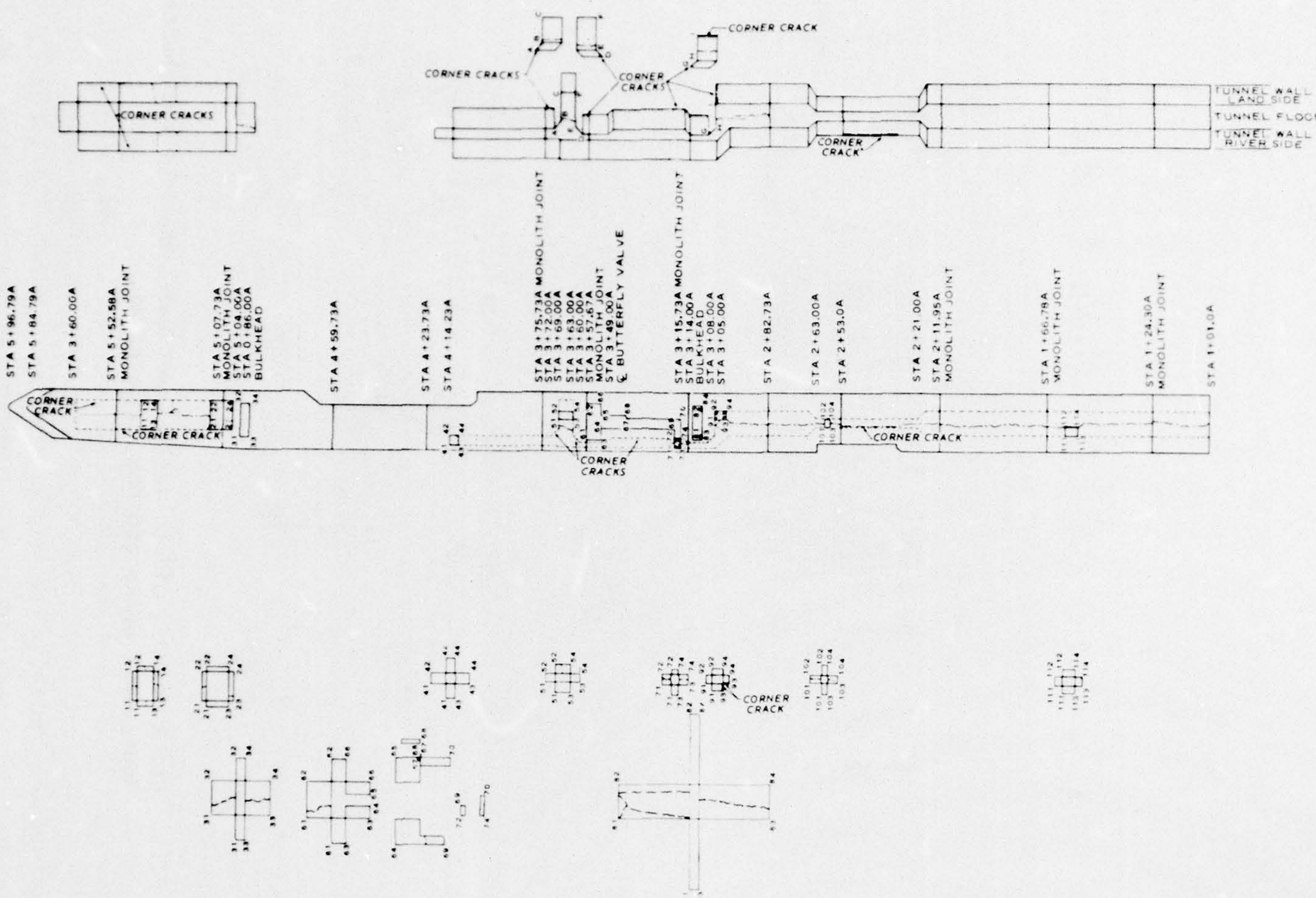


Figure 2.10. Crack survey, middle wall--upper half, tunnels and shafts, Montgomery Locks and Dam, Ohio River.

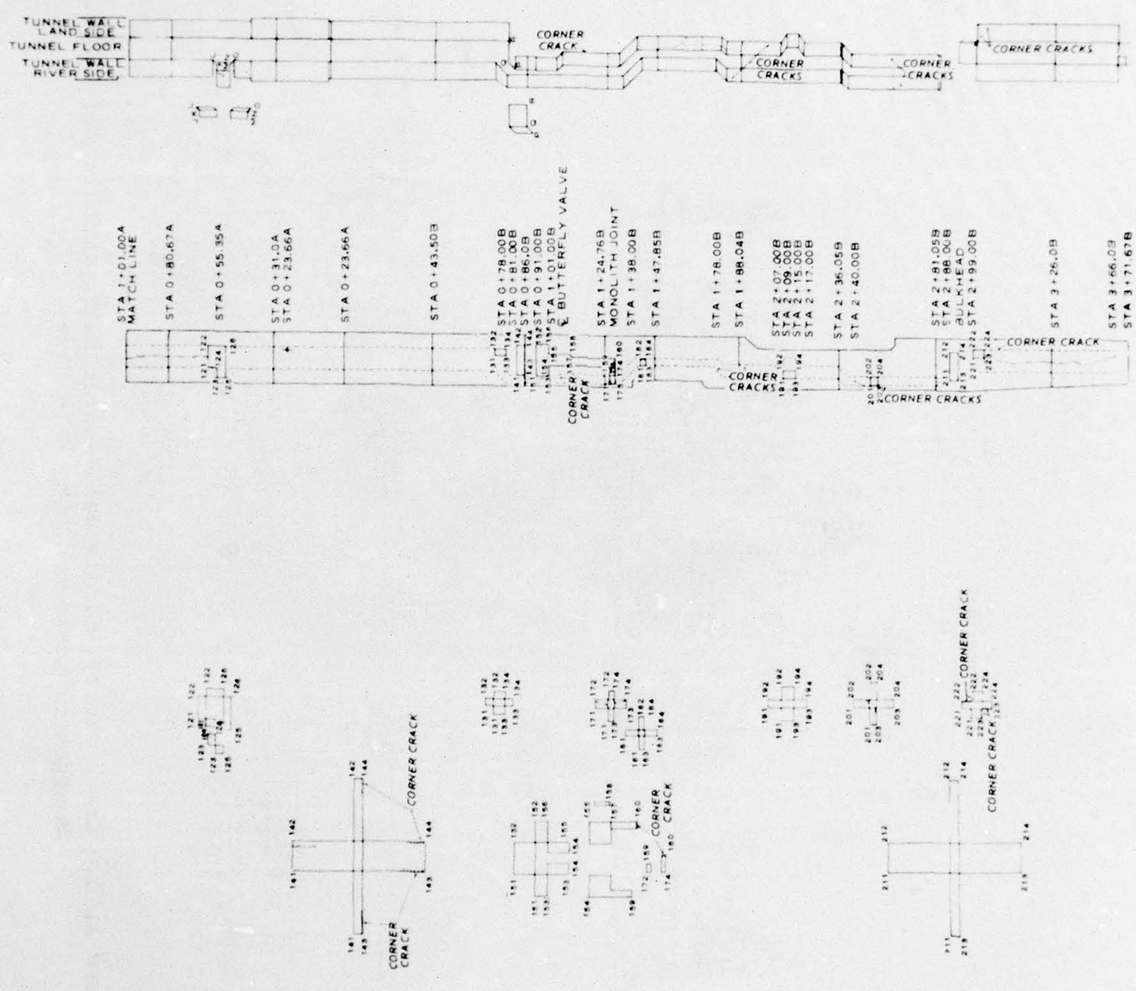


Figure 2.11. Crack survey, middle wall--lower half, tunnels and shafts, Montgomery Locks and Dam, Ohio River.

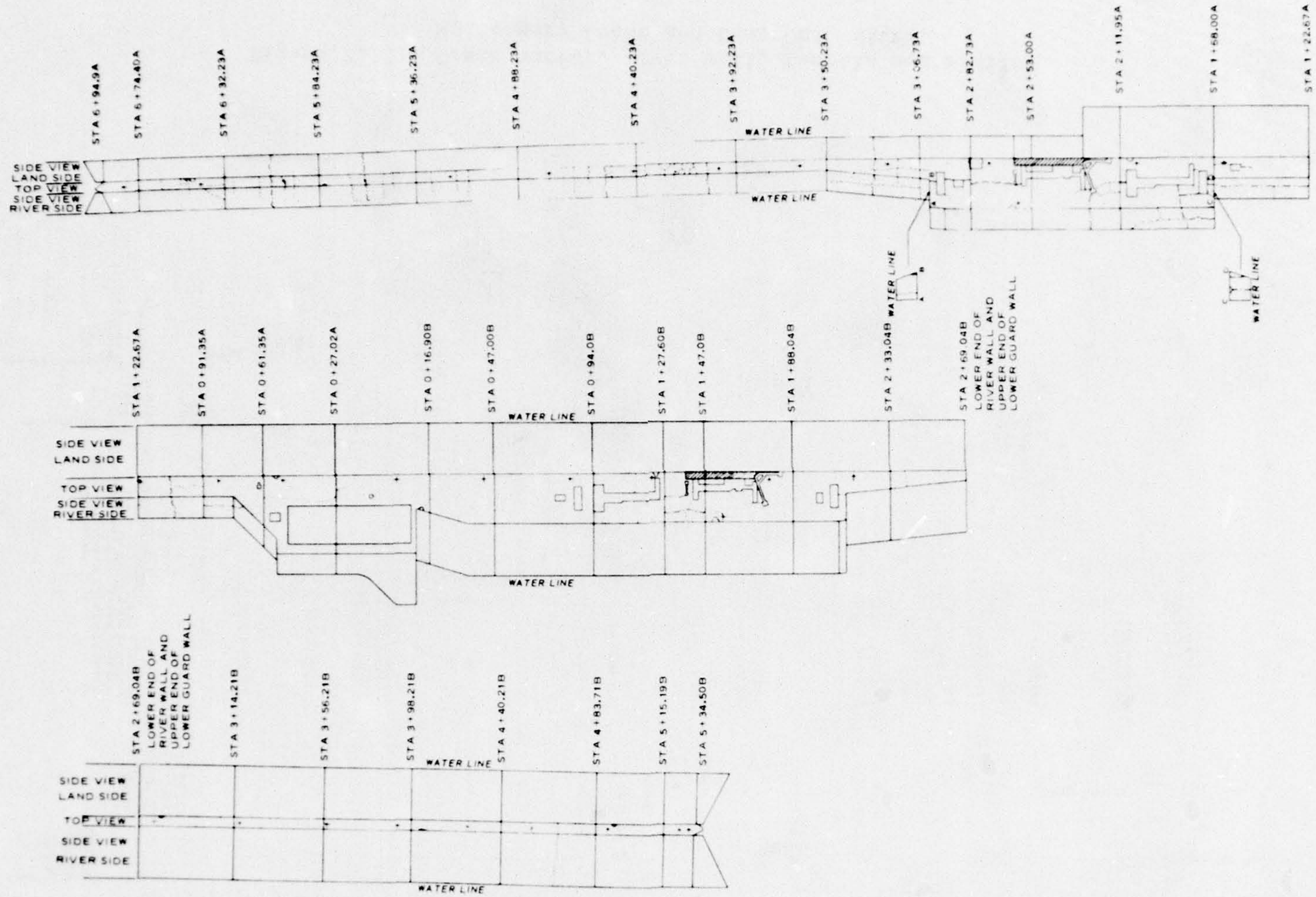


Figure 2.12. Crack survey, river and guard walls, Montgomery Locks and Dam, Ohio River.

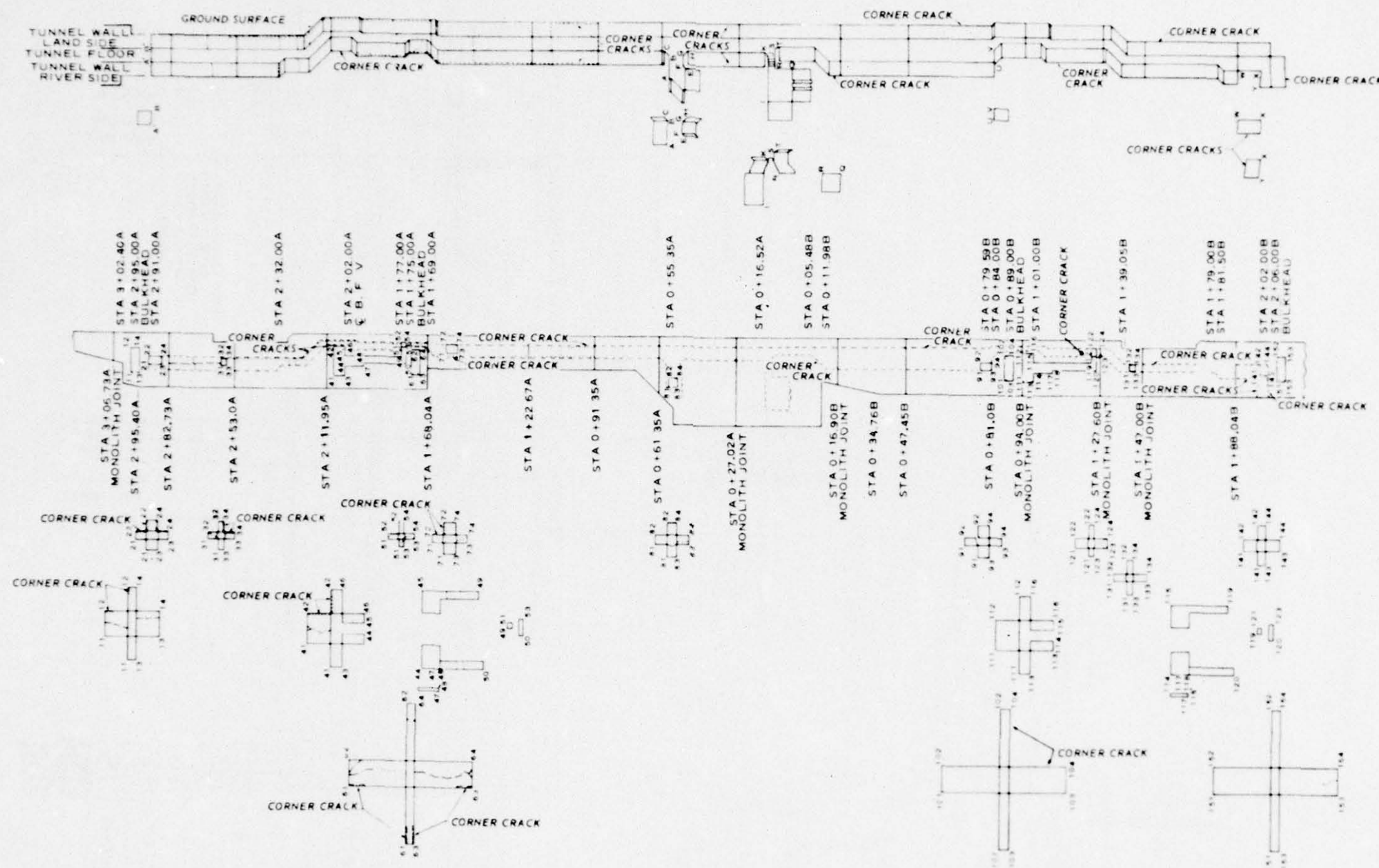


Figure 2.13. Crack survey, river wall, tunnels and shafts, Montgomery Locks and Dam, Ohio River.

Table 2.1

Surface Crack Delineations

Designation	Surface Width, in.	Depth as Can be Seen at the Concrete Surface
....	Width < 1/32	No extent could be seen
-----	Width $\geq$ 1/32 < 5/32	No extent could be seen
___ _ ___	Width < 5/32	Extent seen
	Width $\geq$ 5/32	Extent seen

Table 2.2

Pulse Velocity Test Results, Montgomery Locks and Dam

Test Location	Vertical Distance From Top of Lock Surface (ft) Borehole	Pool	Path Length (ft)	Pulse Velocity (fps)
Landside Wall -	12	12	20.1	None
Borehole to Lock	15	15	"	14,056
wall	20	20	"	13,720
"	25	25	"	14,205
"	30	30	"	Thru
"	35	35	"	Gallery
"	40	40	"	"
"	42	42	"	14,155
"	45	"	20.32	14,411
"	50	"	21.63	14,184
"	55	"	23.94	14,166
"	60	"	26.98	14,052
"	65	"	30.55	14,176
"	70	"	34.47	14,127
"	75	"	38.64	14,076
"	78	"	41.23	14,000
Landside Wall -	15	15	25.3	14,213
Borehole to Flooding	20	20	"	13,863
Chamber	25	25	"	13,978
	30	30	"	13,386
	35	35	"	13,386
	40	40	"	13,171
	45	42	25.48	14,560
	50	"	26.53	14,418
	55	"	28.44	14,305
	60	"	31.05	14,375
	65	"	34.19	14,276
	70	"	37.74	14,268
	75	"	41.58	14,264
	78	"	44.0	14,239
Landside Wall -	Top Surface	15	21	7,368
Top surface to lock	"	20		None
wall across crack on	"	25	29	7,702
butterfly valve wall	"	30		None
	"	42		None

(Continued)

Table 2.2 (Continued)

Test Location	Vertical Distance From Top of Lock Surface (ft) Borehole	Pool	Path Length (ft)	Pulse Velocity (fps)
Landside Wall -	25	25	20.1	14,010
Borehole to lock face.	25	25	20.98	13,846
(First Measurement -	25	25	23.41	14,585
straight thru.	25	25	26.98	None
Additional measurements - Pool transducer moved upstream in 6 ft increments.	25	25	31.31	None
Landside Wall -	8' below	42		None
Surface of vertical	deck on	35		None
butterfly valve wall	vertical	30	27.47	13,112
across crack	face of	27	25.13	12,888
	butterfly	25	23.66	12,583
	valve wall	20	20.36	12,378
	"	15	17.88	11,918
	"	11.6	16.84	11,817
Landside Wall -				
Deck surface close to borehole across visible cracks to vertical wall at station 3 + 21.2A	On surface near borehole	1.25	17.74	9,885
		on vertical wall	15.45	9,224

(Continued)

Table 2.2 (Continued)

Test Location	Vertical Distance From Top of Lock Surface (ft) Borehole	Pool	Path Length (ft)	Pulse Velocity (fps)
Middle Wall - Borehole to large lock wall	12	12	14.8	None
	15	15	"	14,095
	20	20	"	16,424
	25	25	"	15,417
	30	30	"	Thru
	35	35	"	Gallery
	40	40	"	"
	42	42	"	"
	45	"	15.1	14,733
	50	"	16.82	14,566
	55	"	19.7	14,379
	60	"	23.3	14,296
	65	"	27.35	14,062
	70	"	31.671	13,307
Middle Wall - Borehole to the wall of Flooding Chamber	12	12	11.5	15,092
	15	15	"	14,781
	20	20	"	14,896
	25	25	"	14,650
	30	30	"	Thru
	35	35	"	Gallery
	40	40	"	"
	42	42	"	"
	45	"	11.88	None
	50	"	14.01	"
	55	"	17.36	"
	60	"		"
	70	"		"
Middle Wall - Borehole to small lock wall	12	12	8.6	15,168
	15	15	8.6	14,751
	20	20	8.6	14,905
	25	"	9.95	15,237
	30	"	13.19	Thru
	35	"		Gallery
	40	"		"
	50	"		None
	60	"		None

(Continued)

Table 2.2 (Continued)

Test Location	Vertical Distance From Top of Lock Surface (ft) Pool	Pool	Path Length (ft)	Pulse Velocity (fps)
Middle Lock Wall - Across Lock Wall	12	12	24	None
	15	15	"	14,414
	20	20	"	15,738
	25	25	"	17,266
	30	30	"	Thru
	35	35	"	Gallery
	40	40	"	
Middle Lock Wall - Across Lock Wall (36 ft upstream from Gate house)	12	12	24	None
	15	15	"	None
	20	20	"	14,634
	25	25	"	14,634
	30	30	"	Thru
	35	35	"	Gallery
	40	40	"	"
	42	42	"	14,545
Middle Lock Wall - Across Lock Wall (36 ft upstream from Gate house) using surface transducers	1	1	24	11,137
	1 1/2	1 1/2	24	11,137

(Continued)

Table 2.2 (Concluded)

Test Location	Vertical Distance From Top of Lock Surface (ft) Borehole	Pool	Path Length (ft)	Pulse Velocity (fps)
River Wall - Borehole to small Lockwall	12	12	17.95	None
	15	15	"	13,969
	20	20	"	13,915
	24	24	"	13,750
	30	"	18.93	13,616
	35	"		Gallery
	40	"		Interfer-
	45	"		ence
	50	"		"
	55	"		"
	60	"		"
	70	"		"
River Wall - Borehole to small lock wall downstream from above 13.9'	12	12	25.4	14,350
	15	15	"	13,730
	20	20	"	12,926
	25	25	"	13,368
	30	30	"	14,350
	35	35	"	Thru
	40	40	"	Gallery
	42	42	"	13,730
	45	"	25.58	13,938
	50	"	26.63	13,870
	55	"	28.53	13,850
	60	"	31.13	13,624
	65	"	34.27	13,957
	70	"		None
River Wall - Borehole to River Wall	12	12	2.08	14,751
	20	20	2.08	14,751
	30	30	2.08	13,506

SECTION 3: CONCRETE INTEGRITY

3.1 The main concern at Montgomery Locks and Dams is that barge impact will structurally deteriorate the monoliths by cracking. The second concern is the many longitudinal and transverse cracks and the many spalled areas which will allow access of water to some depth into the concrete thereby causing more and more deterioration by freezing and thawing.

3.2 Some measures need to be taken to allow the most economical and best operational use of the Locks and Dam for an extended period of time (30 to 50 yr). The action to take will depend upon the most feasible alternative of rehabilitation or replacement by considering the total locks and dam situation.

3.3 In any case, if the locks and dam are to be operational for over 6 to 8 yr, the cracks, spalled areas, and deteriorated concrete surfaces should immediately be sealed from the weather or deterioration will progressively cause problems which will make early replacement necessary.

3.4 Monitoring plugs should be placed across some of the major cracks to see if there is any movement as the operation of the locks is continued. The more predominate cracks in the land wall upstream butterfly-valve monolith, cracks along the center of the middle wall, and cracks in the upstream river-wall monoliths should be monitored.

3.5 One of the main concerns is the major longitudinal crack down the middle wall which is hypothesized to be formed by barge impact. While this may cause the cracking to be isolated to the height above the filling and emptying culverts, the cracks can progressively become worse, cause the upper part of these monoliths to deteriorate structurally, and cause operation problems. The pulse velocity work indicates that the longitudinal cracking in the middle wall is probably confined to the upper part of the monoliths.

3.6 The main question is whether to replace the locks and dam, make permanent repairs, or to merely repair the surfaces to protect subjacent concrete from the weather. This consideration can only be answered by a feasibility study of either:

a. Maintenance with expected replacement when needed as determined by periodic inspection.

b. Complete rehabilitation.

c. Replacement of the structure.

It may be more economical to repair the deteriorated surfaces in order to add life to the structures and delay replacement until periodic inspections show that the feasibility breakpoint has been reached and replacement is necessary.

3.7 Any final decision concerning remedial measures for the locks and dam should be deferred until the feasibility study has been made.

SECTION 4: LABORATORY TESTS

Material Properties

4.1 The gravity walls are supported on competent rock; therefore, the "at rest" pressure coefficient should be used for obtaining horizontal pressures. It may be that the actual horizontal pressure coefficient is lower than the "at rest" value, but the only way to get actual values is to make a number of tests at the lock and dam site. The scope of this work in time and funding is not such that this type of testing is possible. More discussion concerning the selection of the horizontal pressure coefficient is given in Appendix A. The unit weight of the backfill is given in Table 4.1.

4.2 The concrete properties were obtained from cores. The tests yielded the following information:

- a. Compressive strength, q_u, and densities.
- b. Modulus of elasticity, E.
- c. Poisson's ratio, μ.
- d. Shear modulus, G.

4.3 The densities for the foundation rock cores were obtained using measured volumes and weights. The average value is given in Table 4.1. The unconfined and triaxial compression test specimens were prepared according to standard method of test for triaxial strength of undrained rock core specimens, CRD-C 147.[3] The specimens were cut with a diamond-blade saw and the cut surfaces were ground flat to 0.001 in.; specimens were checked for parallel ends and the perpendicularity of ends to the axis of the specimen. Two vertically and three horizontally mounted linear potentiometers, respectively, were used to measure the vertical and diameter change during compression testing. The displacement measurements were then used to calculate the axial strain, ε_a, and the diametric strain, ε_d. The modulus of elasticity, Poisson's ratio, and shear modulus were calculated from the stress-strain data. Axial specimen load was applied with a 440,000-lb-capacity universal testing machine. Confining pressure for the triaxial compression test was applied by a hand-operated electrohydraulic pump.

4.4 The direct shear test specimens were prepared according to applicable portions of the standard method of test for shear strength, CRD-C 90.[3] The direct shear tests on intact shale were conducted using normal loads, σ_n, of 33, 66, and 100 psi. Tensile test specimens were prepared according to standard method of test for splitting tensile strength of concrete specimens, CRD-C 77.[3]

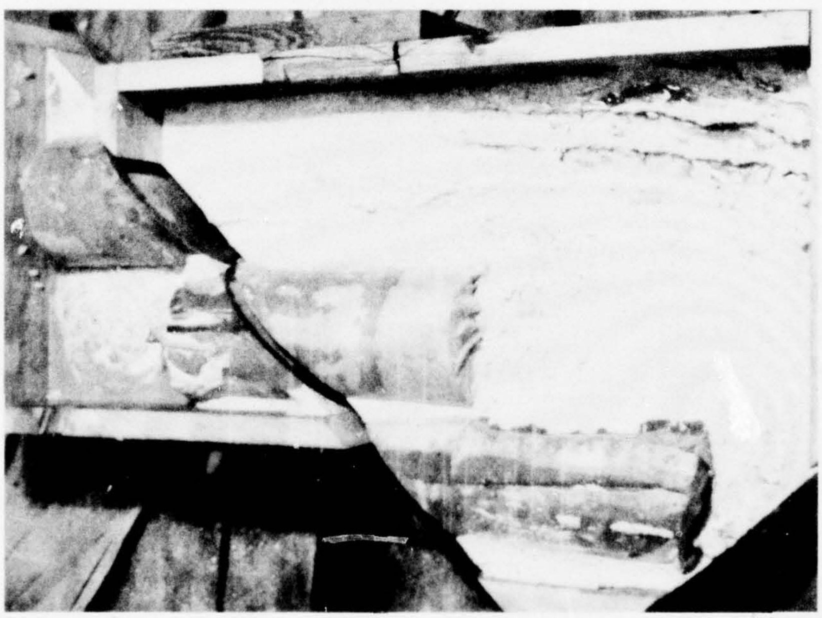
4.5 At the concrete-foundation rock interface it is required to know the coefficient of sliding friction and the cohesion. A multistage triaxial test was conducted to obtain these values.

4.6 The multistage triaxial test was run in the same pressure chamber as the standard triaxial tests. The weights of the piston, swivels, and specimen end platens were accounted for in obtaining the axial load on the specimen. Seven stages were run, including confining (σ_3) pressures of 10, 35, 65, 105, 150, 200, and 300 psi. The sawed surfaces were oriented at an angle of 45 deg from the longitudinal axis of the core.

4.7 Figure 4.1a depicts the orientation of the cores and the method used to cut the cores to insure that the surfaces would reasonably match. The cores were aligned parallel to each other and located relative to each other such that the required portions of the concrete and rock would be obtained. They were then hydrostoned in a wooden box. Figure 5.1b shows the two cores after the 45-deg saw cut was made. When the specimens were removed ɪrom the hydrostone, the concrete and shale surfaces were checked for alignment and found to match quite well; when held to the light, you could only see through about 10 percent of the contact area.

4.8 Some concrete and rock core logs are given in Reference 2 along with a discussion of the petrographic analysis of the concrete and rock material.

a. Core layout.



b. Cut cores matched.

Figure 4.1. Orientation of cores for cutting parallel surfaces.

Table 4.1 Material Parameters

	Foundation	Concrete	Backfill
Index Properties			
Drained Density, lb/ft^3			120.7
Saturated Density, lb/ft^3	163.4	151.3	137.9
Submerged Density, lb/ft^3			75.4
Compressive Strength, psi	3150	6700^+	
Tensile Strength, psi	340		
Shear Strength, psi			
Intact	C = 0 psi $\phi = 71^\circ$		
Concrete on Rock	C = 0 psi $= 30^\circ\ 30'$		
Modulus of Elasticity x 10^6 psi	0.667	5.0	
Poisson's Ratio	0.24	0.25	
Shear Modulus x 10^6 psi	0.269^* 0.377^{**}	1.82	

+ From 1974 Condition Survey Report

* $G = \frac{E}{2(1+\nu)}$

** From TX Data

SECTION 5: STABILITY ANALYSIS

Introduction and Problem Statement

5.1 One main consideration in determining the structural adequacy of the locks and dam is the stability of the various monoliths when subjected to possible loading conditions. The stability study involved analyzing selected monoliths of the locks and dam to determine if there is adequate resistance against overturning, sliding, and base pressures. In this study, only one monolith of each typical configuration and loading was analyzed. The conclusions determined from these data are adequate for an evaluation of all monoliths.

5.2 In addition to the condition survey report,[2] a field survey and examination of Montgomery Locks and Dam were conducted. From the field survey and examination, no relative settlement or misalignment of monoliths was detected. Bench marks and alignment plugs have recently been installed on the locks; therefore, alignment and settlement can be monitored and any movement detected. The resurfacing on most of the monoliths is deteriorating and in many places it is already absent from the concrete surface. The concrete surfaces are badly deteriorated and will be a concern in areas of stress concentrations which will be discussed in the next section.

5.3 The objective of the stability analysis is to determine whether or not the monolith of this old structure meets the present day criteria of desired safety against overturning, sliding, and excessive base pressures. The present day criteria is set forth by the Office of the Chief of Engineers in their Engineering Manuals and Technical Letters. These are the standards which are set forth to reflect the current state of art for the design or analysis of civil works structures. Any advance in the state-of-art reflecting needed changes in these criteria is a separate consideration and is to be used only by approval of the Office of the Chief of Engineers. Even with the criteria from the Engineering Manuals and Technical Letters engineering judgment will have to be used in certain aspects of the analysis. In these considerations

it is important not to relax engineering concepts to include variables which are not dependable because during infrequent but special conditions they could cause failure.

5.4 The first information needed in order to start a stability analysis is the physical geometry of the various monoliths. This is needed in the actual analysis as well as in the selection of the monoliths to be analyzed. When analyzing an old lock, it is important to determine the as-built construction. In this case no as-built plans were available; therefore, other means had to be used to determine construction variations from that planned. Construction photographs and borehole data were used to help establish any construction variations from that which was planned.

5.5 After the monoliths for analysis have been selected and their geometry determined, possible loading conditions must then be determined. A summary of the loading and criteria used in the stability analysis is given below and a more detailed explanation is given in Appendix A.

5.6 The surface elevations of normal upper and lower pools are 682.0 and 664.5, respectively. The saturation levels in the backfill were used as given in Table A-2. These are the levels which have been used as design standards by the Pittsburgh District. The density of backfill material was 120.7 and 137.9 lb/ft^3 for the drained and saturated weights, respectively. The horizontal force expected by the backfill material on the land-wall monoliths was used as a coefficient times the vertical soil pressure at that depth. A lower bound "at-rest" coefficient of 0.5 was used. The location of the resultant soil pressure was considered to be as suggested in Engineer Manual EM 1110-2-2602 for walls supported on rock foundations:

a. 0.38H above the base for horizontal or downward sloping backfill.

b. 0.45H above the base for upward sloping backfill.

5.7 The density of concrete was used as 151.3 lb/ft^3 which was an average of many measurements obtained from cores. The boat impact was:

a. Lock chamber wall: 800 lb/ft but not less than 40,000 lb per monolith.

b. Other walls: 2500 lb/ft but not less than 120,000 lb per monolith.

The hawser pull was considered as 24,000 lb distributed over a monolith of 30 ft length. The boat impact and hawser pull is considered as acting 5 ft above the waterline and is combined with the most severe normal loading conditions.

5.8 An allowable base pressure of 20 K/ft^2 was used. This allowable valve was used because it is that which has become acceptable by the Pittsburgh District. A wind loading of 30 lb/ft^2 was used when applicable. For sliding the cohesion valve (C) was 0 and the angle of sliding friction (ϕ) between the concrete and foundation was 30.5 deg.

5.9 Resistance to overturning was considered adequate if the resultant fell outside the kern but within the middle half of the base for both the normal operation and extreme maintenance cases using "at rest" earth pressure coefficients.

5.10 The criteria for determining resistance to sliding are given in ETL 1110-2-184 and the safety factors are listed in ETL 1110-2-22.

Results

5.11 A summary of the results analysis is given in Table 5.1. A discussion of the stability of the individual monoliths will be given below. Since the inadequacy of the monoliths is the factor which is significant, the monoliths which are inadequate in stability are the only ones which will be discussed.

a. Monolith L-17 is very inadequate for overturning and base pressures for both the normal operation and maintenance cases.

b. Monolith L-19 is very inadequate for overturning and base pressures for both the normal operation and maintenance cases.

c. Monolith L-25 is very inadequate for overturning and base pressures for both the normal operation and maintenance cases.

d. Monolith L-33 is very inadequate for overturning and base pressures for both the normal operation and maintenance cases.

e. Monolith L-42 is supported by a piling foundation which is inadequate in design. The horizontal loads per pile are 14.14^K which is greater than 8^K allowable. The vertical load per pile is 136.7^K which is greater than 100^K allowable.

f. Monolith M-5 is inadequate for sliding in the normal operation case. It is inadequate for sliding and base pressure in the dewatered case.

g. Monolith M-7 is inadequate for overturning in both the normal operation and maintenance cases. It is inadequate for base pressures in the maintenance case.

h. Monolith M-10 is inadequate for overturning and base pressures in the maintenance case.

i. Monolith M-13 is inadequate for overturning in both the normal operation and maintenance cases. It is inadequate for base pressures in the maintenance case.

j. Monolith M-22 is somewhat inadequate for overturning in the normal operation case.

k. Monolith R-12 is inadequate for overturning and base pressures in the maintenance case. It is very inadequate for sliding in the maintenance case.

l. Monolith R-13 is inadequate for overturning in both the normal operation and maintenance cases. It is inadequate for base pressures in the maintenance case.

m. Monolith R-15 is inadequate for overturning for the normal operation and maintenance cases. Base pressure is a little high for the dewatered case.

n. Monolith R-20 is inadequate for overturning in the normal operation case and is a little inadequate for sliding in the maintenance case.

o. Monolith R-23 is a little inadequate for overturning in the normal operation case.

p. The lower miter sills are inadequate for sliding in the maintenance cases.

5.12 An analysis of typical dam piers is given in Reference 1 which shows that they are stable; these results are not reproduced in this report.

5.13 Almost all of the monoliths in the land wall are inadequate in their resistance to overturning and base pressures. Under unfavorable conditions this could cause problems.

5.14 In general the monoliths in the middle and river walls are inadequate in their resistance to overturning. This is especially true for the middle wall monoliths in the dewatered case. If the locks are dewatered, precautions would have to be taken to keep the middle wall monoliths stable.

5.15 Monoliths M-5, M-10, and R-12 are inadequate for sliding in the normal operation case which could allow them to experience movement when subjected to unfavorable conditions.

5.16 The miter sills are inadequate for sliding if the locks are dewatered.

5.17 The middle and river wall monoliths are close enough to be considered adequate for base pressures.

5.18 There are two acceptable approaches to this situation when considering only the stability of monoliths. One approach is to say the monoliths do not meet the criteria and examine the feasibility of modifying the construction or replacing the locks and dam. The other approach is to give consideration to the length of time the monoliths have been in service without excessive relative settlement or misalignment, and to schedule periodic inspections of the locks and dam so that any potential trouble can be detected and corrective action taken. The periodic inspection has merit because minimum maintenance can be performed to protect the monoliths from weathering, and decisions of replacement made when conditions warrant such action. Rehabilitation or replacement should be considered, taking into account the total condition (operational and structural) of the locks and dam.

Table 5.1

Summary of Stability Analysis Results

Monolith	Cases Considered	Percent Effective Base: Minimum Allowable	Percent Effective Base: Actual	Sliding Safety Factor: Minimum Allowable	Sliding Safety Factor: Actual	Foundation Pressure, k/sf: Allowable	Foundation Pressure, k/sf: Actual
L-5	Normal operation with impact Normal operation with hawser	Pile loads are below allowables (see Detail Calculations in Appendix A)					
L-17	Normal operation with gate load	75	11	4	1.65	20	120
	Maintenance	75	0	2-2/3	1.09	20	∞
L-19	Normal operation	75	16	4	1.66	20	81
	Maintenance	75	0	2-2/3	1.31	20	∞
L-25	Normal operation	75	16	4	1.66	20	80
	Maintenance	75	2	2-2/3	1.30	20	765
L-33	Normal operation	75	0	4	1.47	20	∞
	Maintenance	75	0	2-2/3	1.07	20	∞
L-42	Normal operation	Pile loads are above allowables (see Detail Calculations in Appendix A)					

(Continued)

Table 5.1 (Continued)

Monolith	Cases Considered	Percent Effective Base		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
M-5	Normal operation	100	100	4	9.03	20	8.7
	Maintenance	75	47	2-2/3	1.24	20	27.4
M-7	Normal operation	100	76	4	2.67	20	13.7
	Maintenance	75	44	2-2/3	1.12	20	28.0
M-10	Normal operation	100	100	4	3.68	20	11.2
	Maintenance	75	58	2-2/3	1.24	20	22.7
M-13	Normal operation	100	75	4	1.83	20	13.7
	Maintenance	75	45	2-2/3	1.06	20	28.6
M-20	Normal operation	100	100	4	2.36	20	9.4
	Maintenance	75	77	2-2/3	1.21	20	16.5
M-22	Normal operation	100	71	4	1.89	20	15.9
	Maintenance	75	100	2-2/3	2.44	20	11.7

(Continued)

Table 5.1 (Concluded)

Monolith	Cases Considered	Percent Effective Base: Minimum Allowable	Percent Effective Base: Actual	Sliding Safety Factor: Minimum Allowable	Sliding Safety Factor: Actual	Foundation Pressure, k/sf: Allowable	Foundation Pressure, k/sf: Actual
R-6	Normal operation	Pile loads are below allowables (see Detail Calculations in Appendix A)					
R-12	Normal operation	100	100	4	11.10	20	5.7
	Maintenance	75	55	2-2/3	1.10	20	23.5
R-13	Normal operation	100	82	4	2.96	20	13.8
	Maintenance	75	48	2-2/3	1.30	20	28.3
R-15	Normal operation	100	88	4	2.61	20	11.2
	Maintenance	75	47	2-2/3	1.13	20	25.3
R-20	Normal operation	100	71	4	1.33	20	15.2
	Maintenance	75	100	2-2/3	3.06	20	12.8
R-23	Normal operation	100	90	4	1.55	20	12.6
	Maintenance	75	92	2-2/3	2.90	20	15.0
R-29	Normal operation	Pile loads are below allowables (see Detail Calculations in Appendix A)					
Lower miter Sill (110' lock)	Normal operation	100	100	4	2.71	20	3.8
	Maintenance	75	100	2-2/3	6.82	20	7.8
Lower miter Sill (56' lock)	Normal operation	100	99	4	1.97	20	5.0
	Maintenance	75	100	2-2/3	5.95	20	5.5

SECTION 6: STRESS ANALYSIS

Introduction and Problem Statement

6.1 In the structural evaluation of Montgomery Locks, a two-dimensional plane strain finite-element analysis was used to determine stresses within selected structural monoliths.

6.2 It is becoming increasingly important to understand the phenomenon of stress distribution in structures and not depend entirely on average stress approximations as has been done in conventional design. Knowledge of the total stress field is important in order that stress concentrations and decisions for concrete reinforcement can be handled wisely and economically. This is especially important when considering that raw materials are being depleted and should be used wisely and not at a rate in excess of that which is absolutely necessary. Conventional analysis usually leads to a safe but overly conservative design because the whole stress field is not known and observations of stress concentrations cannot be delineated, studied, and adequately reinforced. The finite-element analysis adds a new dimension or advantage in this respect. Finite-element studies do not make conventional design obsolete; in fact, it is a supplement, making a combination which is much better than either separately. It is important to consider stress distribution in areas of stress concentration when evaluating old structures which have cracked and are deteriorating.

Finite Element Analysis

6.3 The finite-element analysis is used to get some idea of the magnitude of compressive- and tensile-stress concentrations within the monoliths under normal operation and maintenance conditions. The finite-element solution gives good results as long as the model adequately represents the actual situation and as long as any assumptions made can logically be seen or proven to be adequate. In the following analysis, elements were made continuous under the monolith which allows tension

between the base and foundation which is, of course, unrealistic. The tension effect dissipates rapidly but will decrease the compressive stresses at the base-foundation interface on the opposite side of the monolith. This effect can be eliminated but the time and funding required to do this trial and error solution are beyond the scope of this project.

6.4 The loads applied on the two-dimensional sections are presented in Figures 6.2, 6.9, 6.15 and 6.19. In a two-dimensional analysis of the monoliths, such factors as changes in geometry and loading along the monoliths lengths can only be approximated. Localized loading (gate anchorages, impact, hawser, etc.) will not give realistic stresses if applied on a one-foot length of monolith. In order to obtain more realistic stress values, concentrated loadings were considered by using a per foot load obtained from distributing the total load over a length or a portion of the length as given by a 45° distribution. The 45° distribution originates at the point of load concentration and extends in the direction of loading until its sides intersect the outer edge of the monolith. This can be done in both the horizontal and vertical planes with the shortest distance between intersections being the more critical. The distance between intersections in the more critical plane was used as the length over which to distribute the concentrated compressive loads and one-half this type distribution was used for concentrated tensile loads. The maximum compressive values were at the intersection of the base and foundation and a 45° distribution will give as reasonable a spread of the load to the foundation as any assumed distribution.

6.5 The maximum tensile stresses are around culverts at changes in geometry, at hawser locations, and at anchorages. The maximum is rather localized at the point of application and will only be relieved by deformation tending to spread the load over the section of concrete which is being separated from the monolith by tension. A 45° distribution of this tensile load will give concentrated stresses which are too low; therefore, an approximation of one-half the 45° distribution was used in the analysis.

6.6 An important concept is that changes in geometry and loading along the monolith length make the problem a three-dimensional analysis and approximations have to be made in a two-dimensional analysis. In the following work, the two-dimensional analysis is used to obtain some idea of maximum stresses in the monoliths. Three-dimensional analysis should be used in a detailed evaluation of stress distribution which is not the objective of this study.

6.7 Average elevations of soil behind the monoliths were considered as was done in the stability analysis. In making stress and displacement calculations, the backfill was not used as part of the grided medium. There were two reasons for this:

> a. Many elaborate tests of backfill material would be required to define the backfill properties precisely. This was not done because the vertical and horizontal backfill loads, which are obtained by using unit weights and coefficient of at-rest-earth pressure, are within the accuracy of the study.
>
> b. The finite-element grid would become very large.

The density of the backfill material was used to get vertical loads. The coefficient of at-rest-earth pressure and the density of the backfill material were used to obtain the horizontal loads. The water pressure from saturation level was taken into account. The loads are then applied at node points of the finite-element grid.

6.8 One consideration which must be made in the stress analysis is the effect of uplift on the base of the monolith. In certain cases the effect will be negligible, but in others it could be substantial; therefore, the effect must be included. The important concept concerning uplift is that it is a support condition, and its effect (small or large) is dependent on its distribution. Specific loadings on a structure cause a specific distribution of pressure under the monolith base. The uplift will change this distribution, thereby affecting the support condition of the monolith. It can be seen that the pressure distribution under the monolith affects the stress in the structure only by deformations resulting from the support condition. By looking at free body force diagrams of a monolith, in fact, a section an infinitesimal distance above the base (in rigid body analysis), can be taken and the

upper part of the monolith considered as a free body. The analysis will then not even see the pressure distribution at the base; therefore, the distribution affects the stress analysis through deformations which are taken into account in the finite-element study. Uplift could have significant effect where there are large culverts close to the base of the monolith and the distribution is such as to load the slab to increase stresses. A reasonable way to handle the uplift is to put a slit of foundation material below the structure of thickness such that the deflection of the monolith at the base is less than the slit thickness in order that problems in code solutions, such as negative element areas, and aspect ratio, will not be encountered.

6.9 The stresses given in the finite-element computations follow the nomenclature given in Figure 6.1 below.

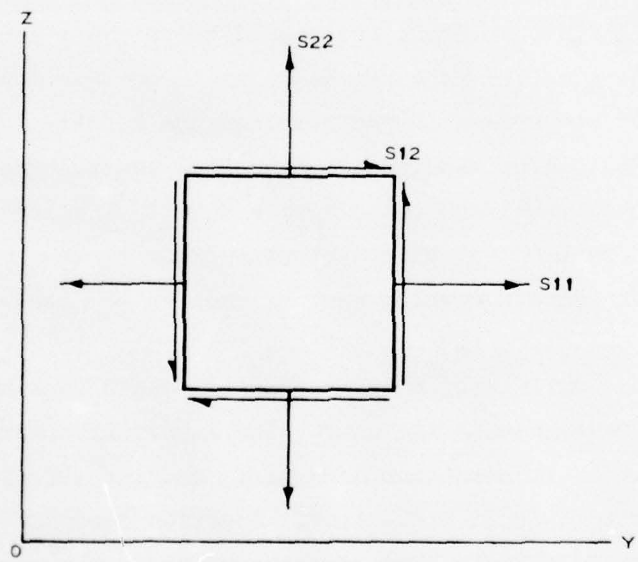


Figure 6.1. Stress nomenclature.

6.10 The stress distributions in this section of the report show the major and minor principal stresses in their respective directions at the centroid of the element. The arrows denote tension if directed toward the centroid, and compression if directed away from it. The values printed on the stress plot are the element number, minimum principal stress, and maximum principal stress. A positive sign indicates tension and a negative sign indicates compression.

6.11 Monoliths within the lock which will have maximum tensile and compressive stresses were selected for analysis.

6.12 Cracking in concrete is a stress problem and in general is not an important factor in the stability of a structure. With this in mind, the most important thing to do is to determine the cause of the cracking and from this, decide if the region of overstressing will continue to exist. This will allow a determination of the importance of the crack and what remedial measures are applicable. For example, the crack may have been caused by past temperature variations and will be significant only in that they should be sealed and protected from the weather. In other cases the cause of cracking may continue and will be a definite threat to the structural integrity of the structure.

6.13 As has been discussed in Section 2 much of the cracking, especially the longitudinal crack down the center of the middle wall, is probably caused by barge impact. Barge impact will continue to be applied to the monoliths even more severely than in the past because of the trend to push heavier loads. This can have a significant effect on the structural integrity of the lock monoliths. The stress analysis results can now be studied and their impact on the cracking accessed. Stress analysis was performed on monoliths L-17, M-8, M-10, and R-12. L-17 is a landside gate bay monolith which, in the dewatered case, will have soil and gate loads producing stresses. Monoliths M-8 and M-10 are in the region of the middle wall where the predominate crack extends down the center of the wall parallel to the lock chamber. R-12 was selected to examine the cracks in a river wall gate bay monolith.

6.14 The loadings for monolith L-17 are given in Figure 6.2. The total stress plots for the normal operation and dewatered cases are

given in Figures 6.3 and 6.4, respectively. The monolith section depicting the area of stress concentration (Area "A") is given in Figure 6.5. The areas showing the stress concentration plates for the normal operating and dewatered cases are given in Figures 6.6 and 6.7, respectively. The concern which first comes to mind is that there could be excessive tensile stresses due to the gate and horizontal soil loads acting on the monolith simultaneously. The gate loads do not predominate; especially, if they are applied over half of a 45° distribution across the section on which they act. In reality at the concentrated position of the anchors, there may be higher tensile stresses than the distribution indicates but as the concrete strains the loads will distribute over a wider area allowing a decrease in maximum stresses. The values obtained are about as good of an approximation as can be obtained without doing a three-dimensional analysis of the monolith.

6.15 The largest tensile stresses are obtained at locations of stress concentration which are around the culverts and at positions of change of shape (where the monolith is stepped). The maximum tensile stresses around the circular culverts are approximately 120 and 290 psi for the normal operational and dewatered cases, respectively; therefore, they are excessive for unreinforced concrete. At the step in the monolith the maximum tensile stresses are about 125 and 150 psi for the normal operational and dewatered cases, respectively, and are also excessive. The analysis shows that the stresses attenuate rapidly as the distance from the area of stress concentration increases. This means that properly placed reinforcement around areas of stress concentration is very important and can eliminate concrete cracking.

6.16 The maximum compressive stresses at the base of the monolith are in excess of 600 and 1250 psi for the normal operational and dewatered cases, respectively.

6.17 The loadings for monolith M-8 are given in Figure 6.8. The total stress plot for the normal operating case is given in Figure 6.9. The monolith section depicting the area ("B") of stress concentration is given in Figure 6.10. The depicted areas of stress concentration are given in Figure 6.11. The tensile stresses for the normal operating

case are as large as those for the dewatered case; therefore, the areas of stress concentration are only shown for the normal operating case. Monolith M-8 is a middle lock wall monolith in the region where the wall is cracked along its center parallel to the lock. The maximum tensile stresses are around the filling and emptying culverts and at the step where the monolith changes shape. The maximum tensile stress around the culvert is somewhat larger than 800 psi. This stress is too large and will crack the concrete. From Figure 6.11 it can be seen that when the concrete cracks tensile stresses are allowed to distribute upward through the center of the monolith. This distribution can distribute through the pipe gallery and generally cause center line cracking down the center of the middle wall. The loads are instantaneous and will have a dynamic load factor in relation to static loads; this will make the instantaneous loading more damaging than a static loading. Computer computations were made with and without barge impact. From a comparison of the stress plots it was seen that barge impact was what substantially causes the large tensile stresses.

6.18 The compressive stresses at the base of M-8 are over 1200 psi and can cause problems in fragmented or deteriorated concrete or foundation material. The foundation material under the lock is fragmented.

6.19 The loadings for monolith M-10 are given in Figure 6.12. The total stress plot for the normal oprating case is given in Figure 6.13. The monolith section depicting the areas ("C" and "D") of stress concentration is given in Figure 6.14. The depicted areas of stress concentration are given in Figures 6.15 and 6.16. Monolith M-10 does not have the impact load applied and the tensile stresses can be seen to be decreased considerably. The maximum tensile stresses are approximately 130 psi and the maximum compression about the same which is much lower than those present when boat impact loads are applied.

6.20 The loadings for monolith R-12 are given in Figure 6.17. The total stress plot for the normal operating case is the more critical and is given in Figure 6.18. The monolith section depicting the areas ("E" and "F") of stress conditions is given in Figure 6.19. The depicted

areas of stress concentration are given in Figures 6.20 and 6.21. The stresses due to gate and other loadings do not produce excessive stresses in this monolith. The cracking in this monolith is small and is at discontinuities or cutouts.

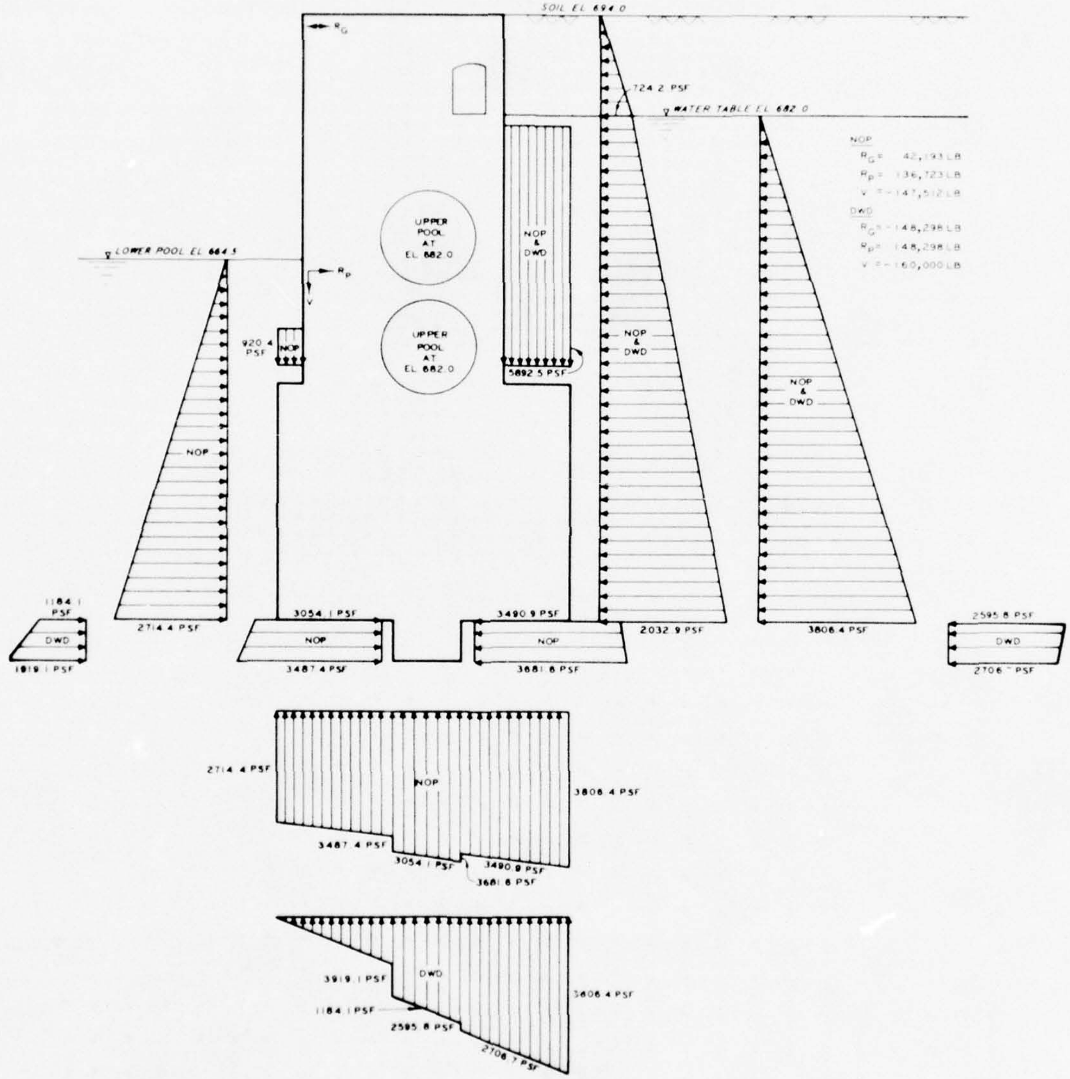


Figure 6.2. Loading, monolith L-17.

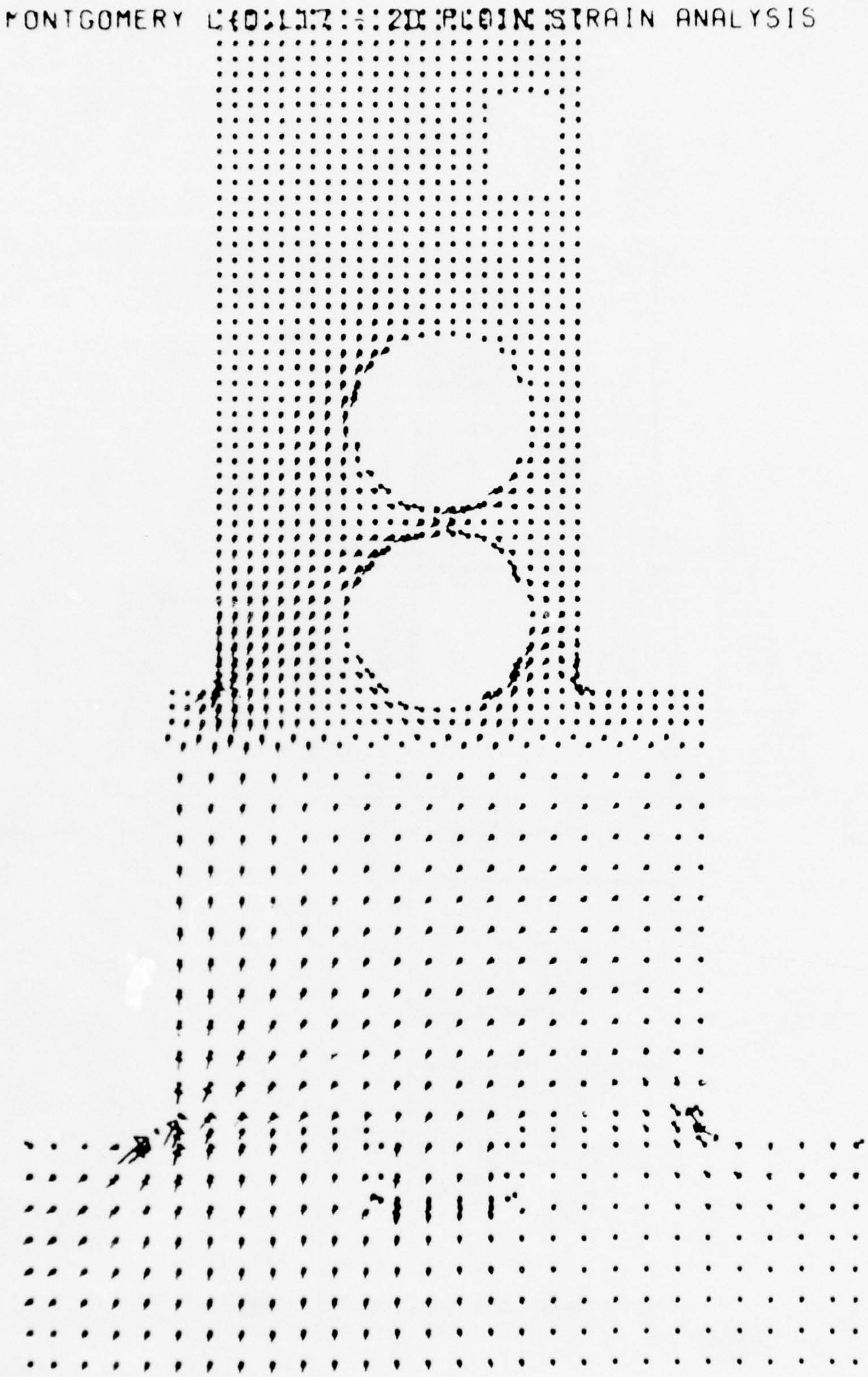


Figure 6.3. Monolith L-17, total stress distribution normal operating case.

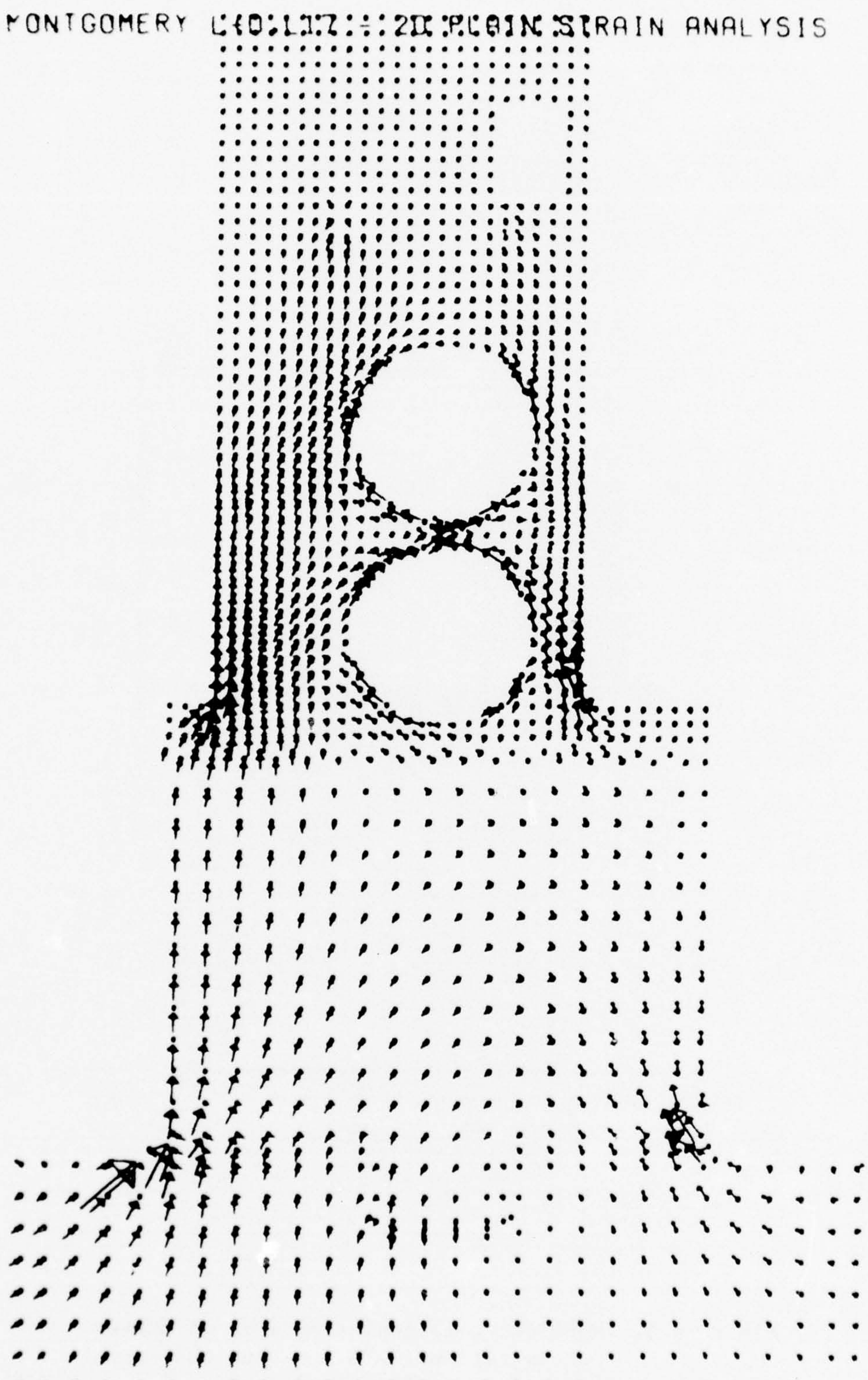


Figure 6.4. Total stress distribution L-17 dewatered.

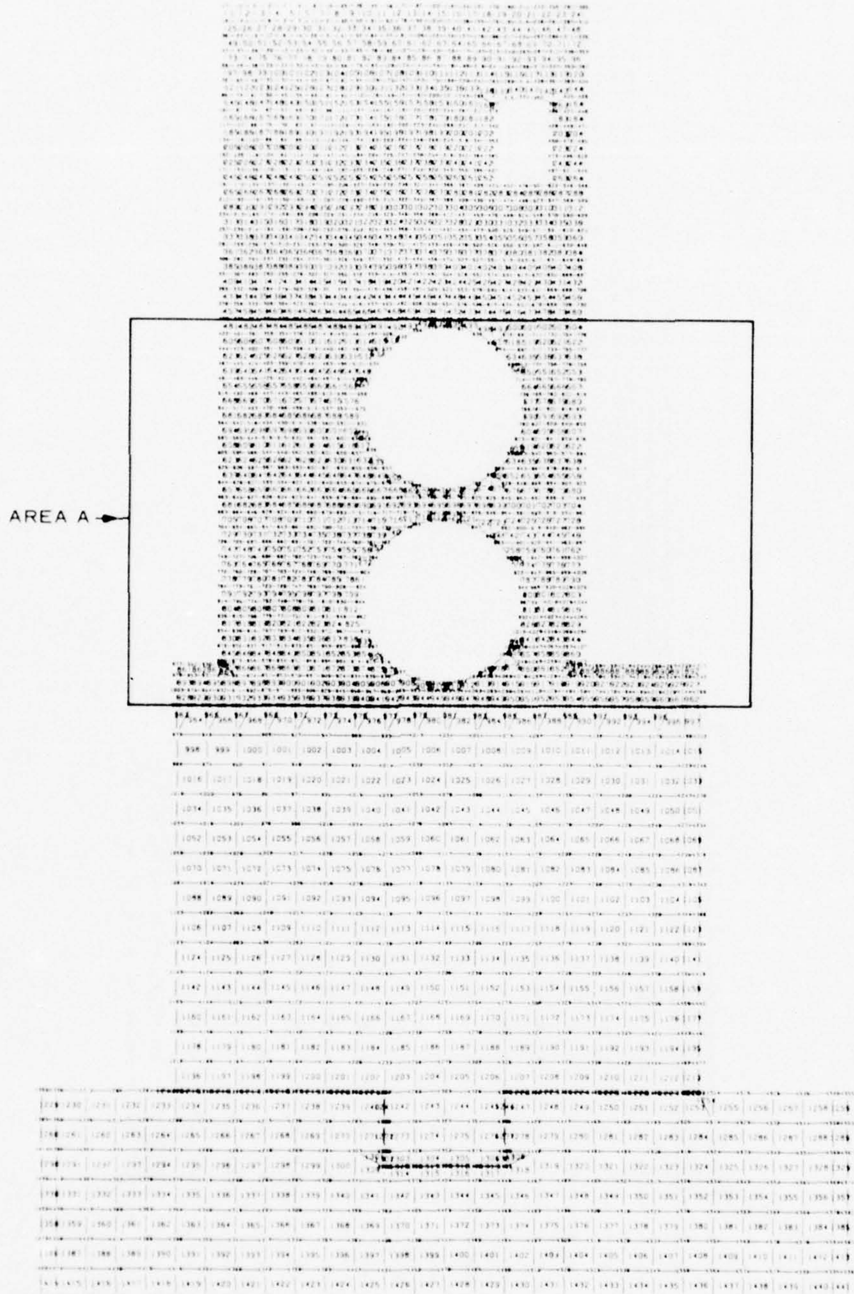


Figure 6.5. Monolith L-17 depicting area of stress concentration ("A") for both the normal operating and dewatered cases.

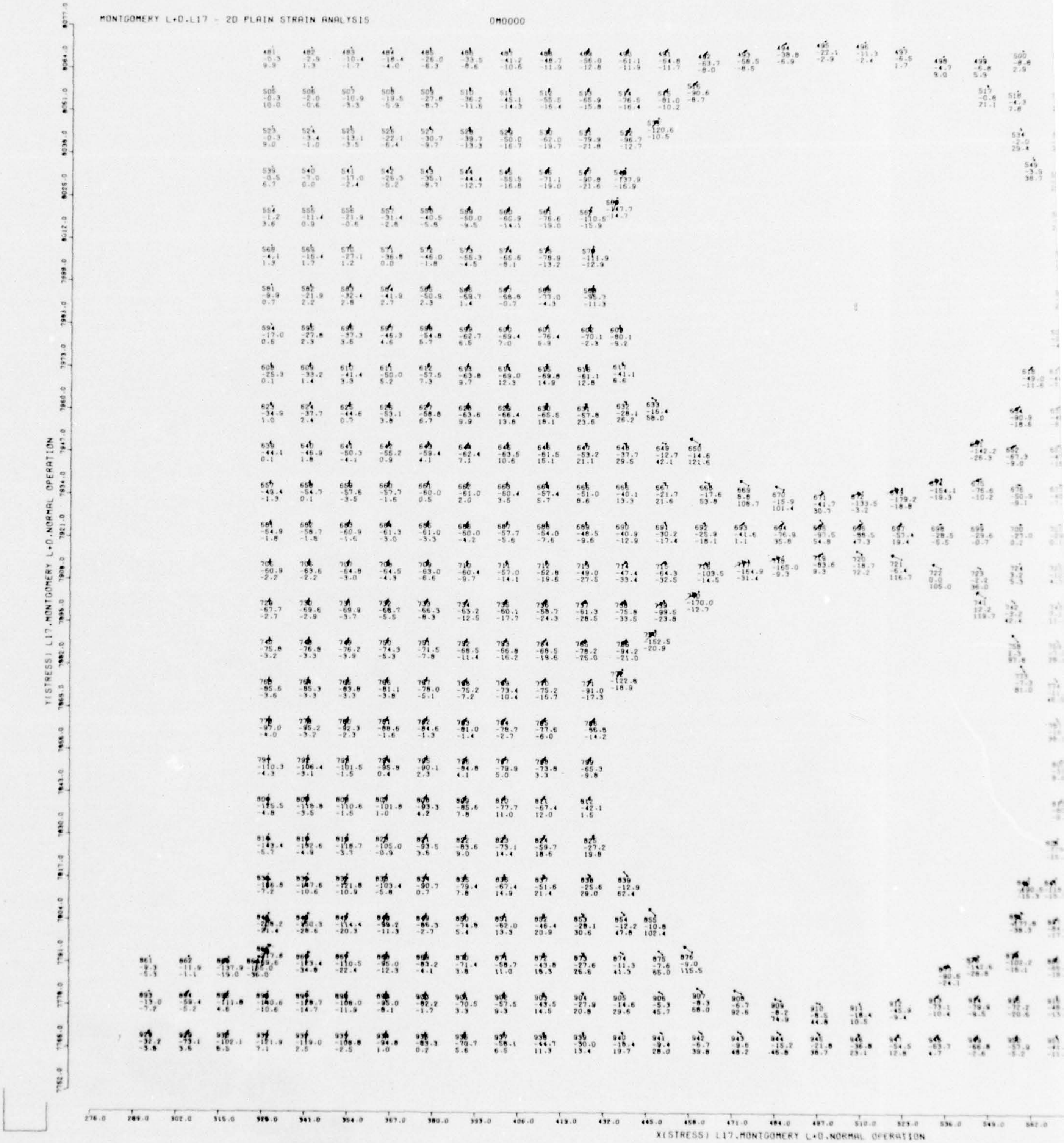
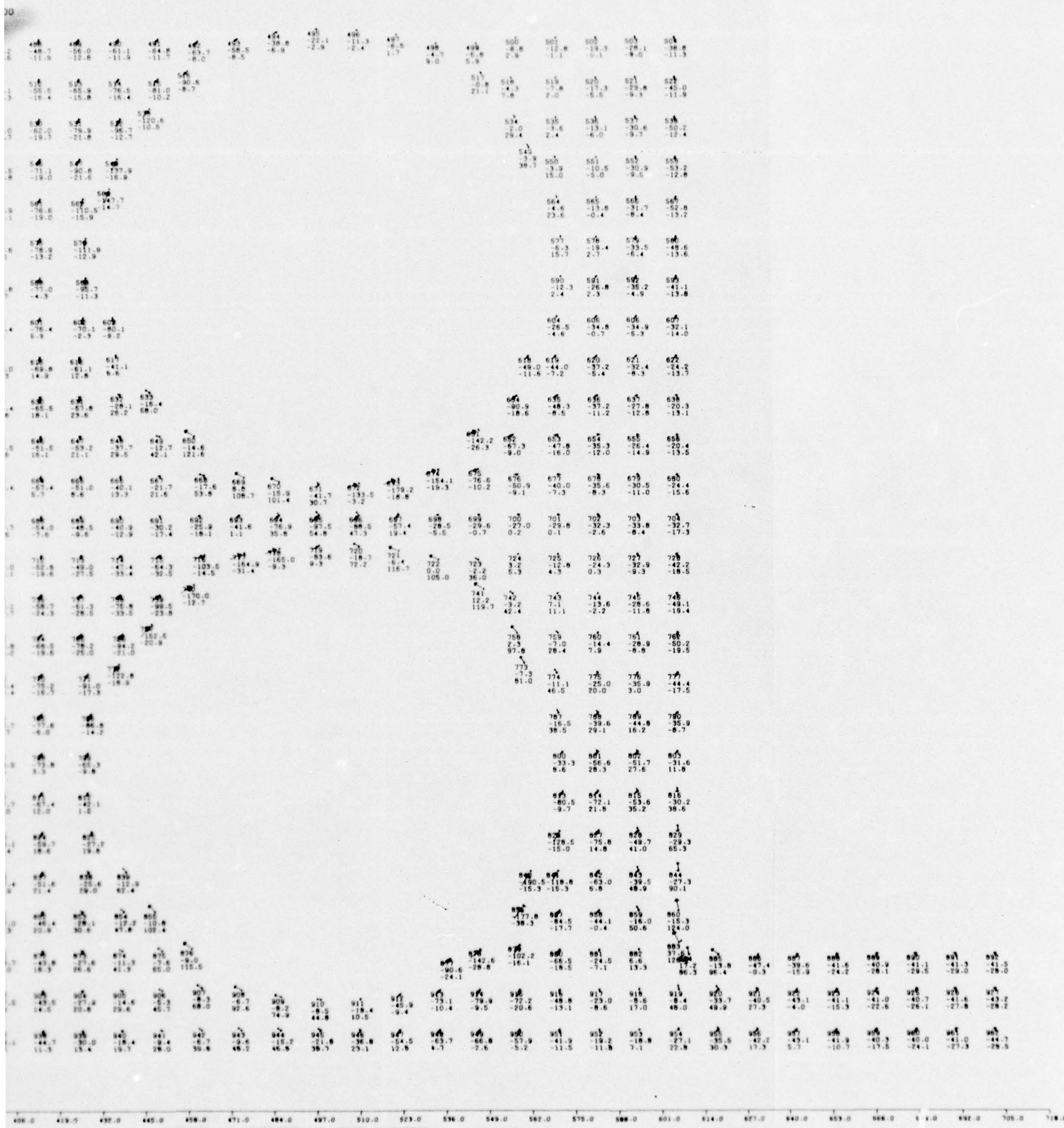


Figure 6.6. Monolith L-17, area of stress concentrations as by area "A" normal operating case.



. Monolith L-17, area of stress concentrations as depicted by area "A" normal operating case.

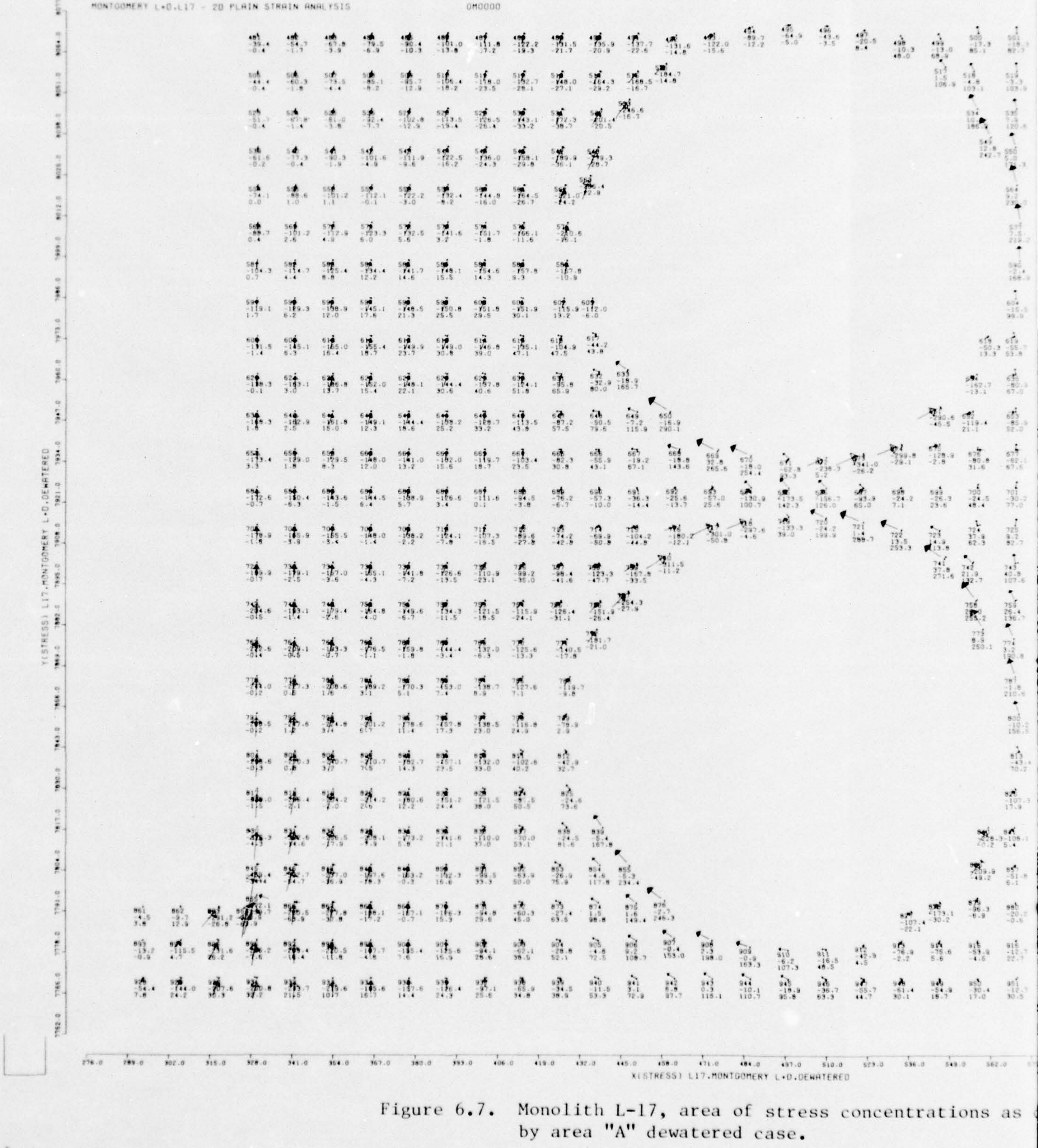
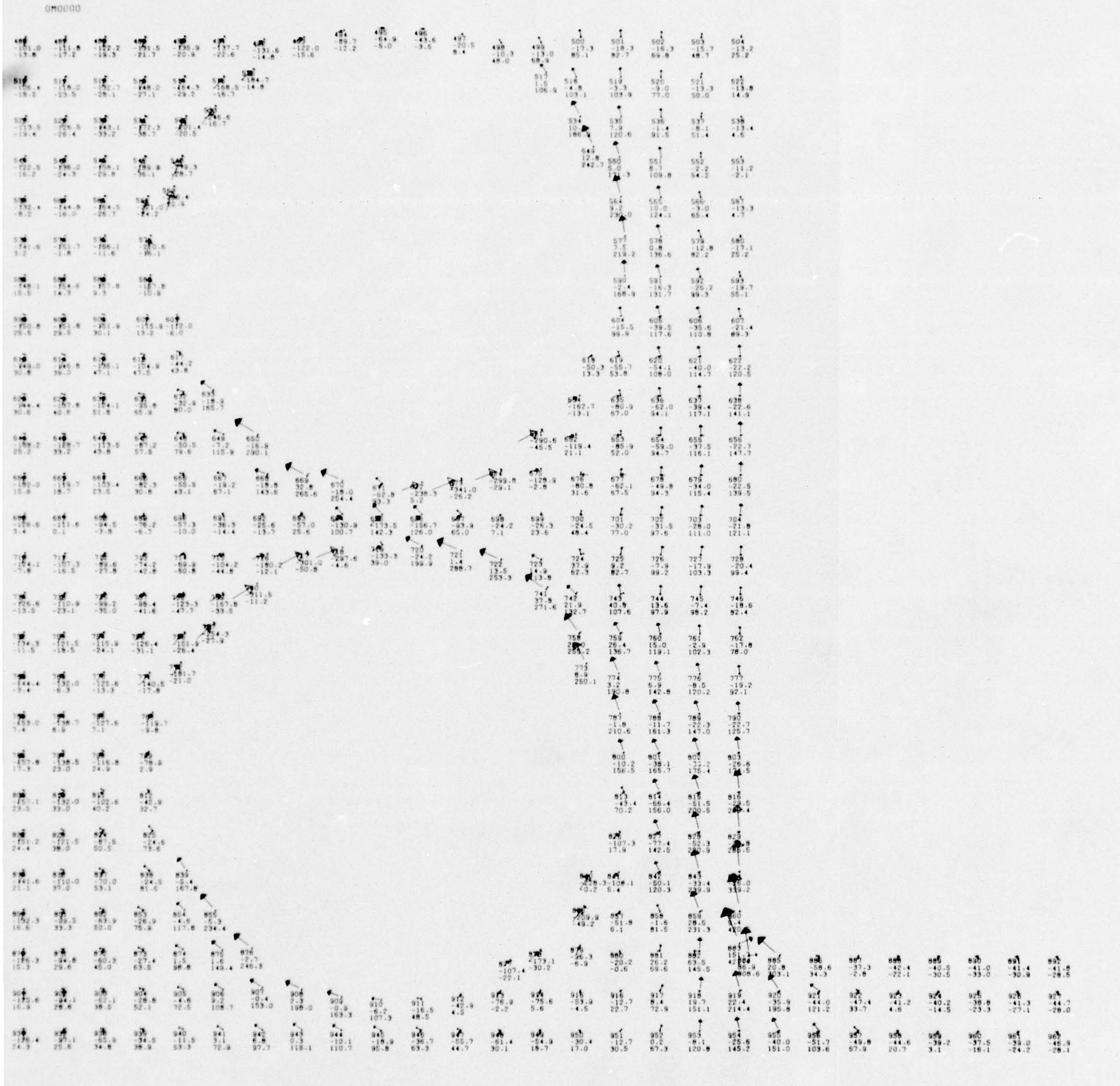


Figure 6.7. Monolith L-17, area of stress concentrations as by area "A" dewatered case.



re 6.7. Monolith L-17, area of stress concentrations as depicted by area "A" dewatered case.

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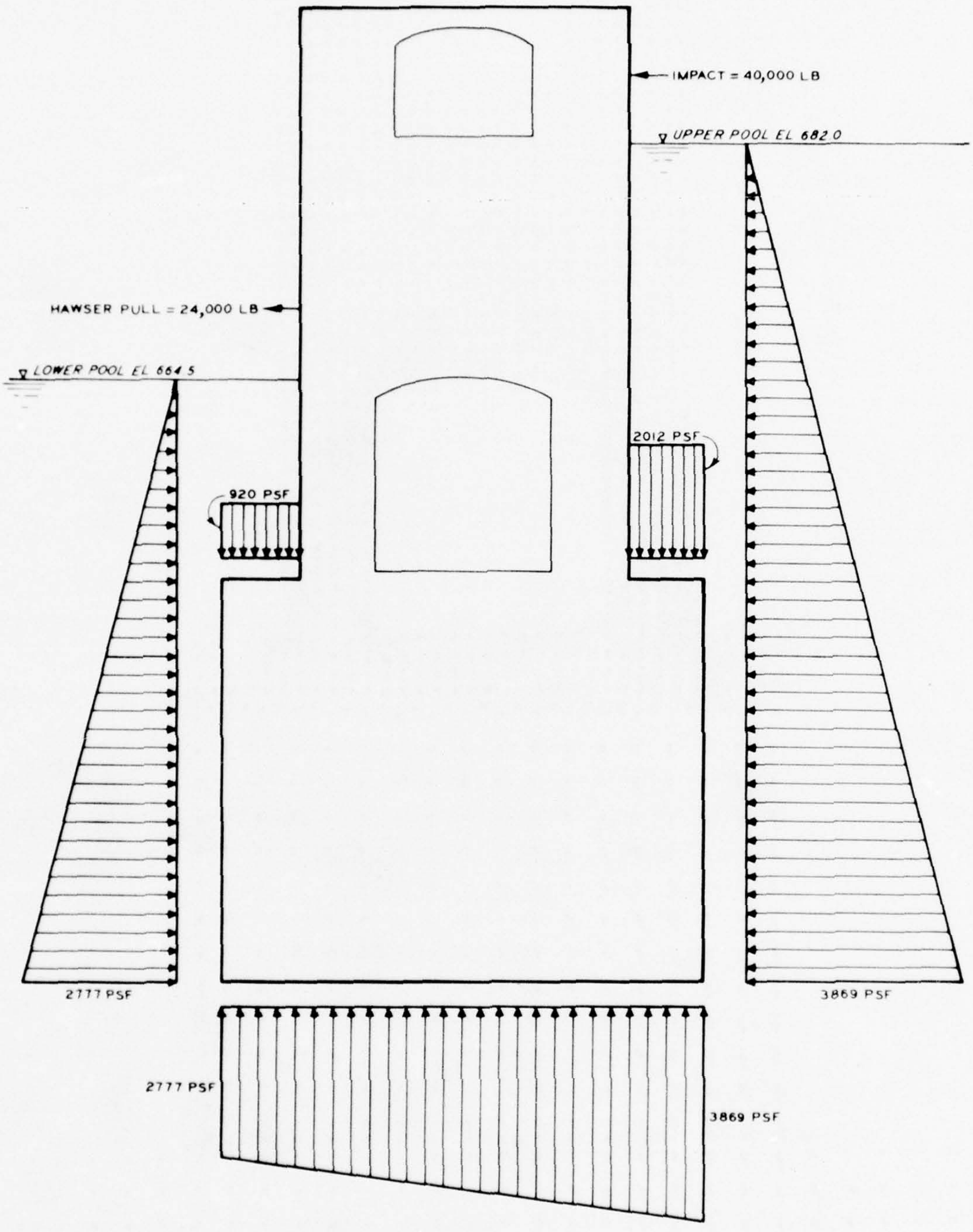


Figure 6.8. Loading, monolith M-8.

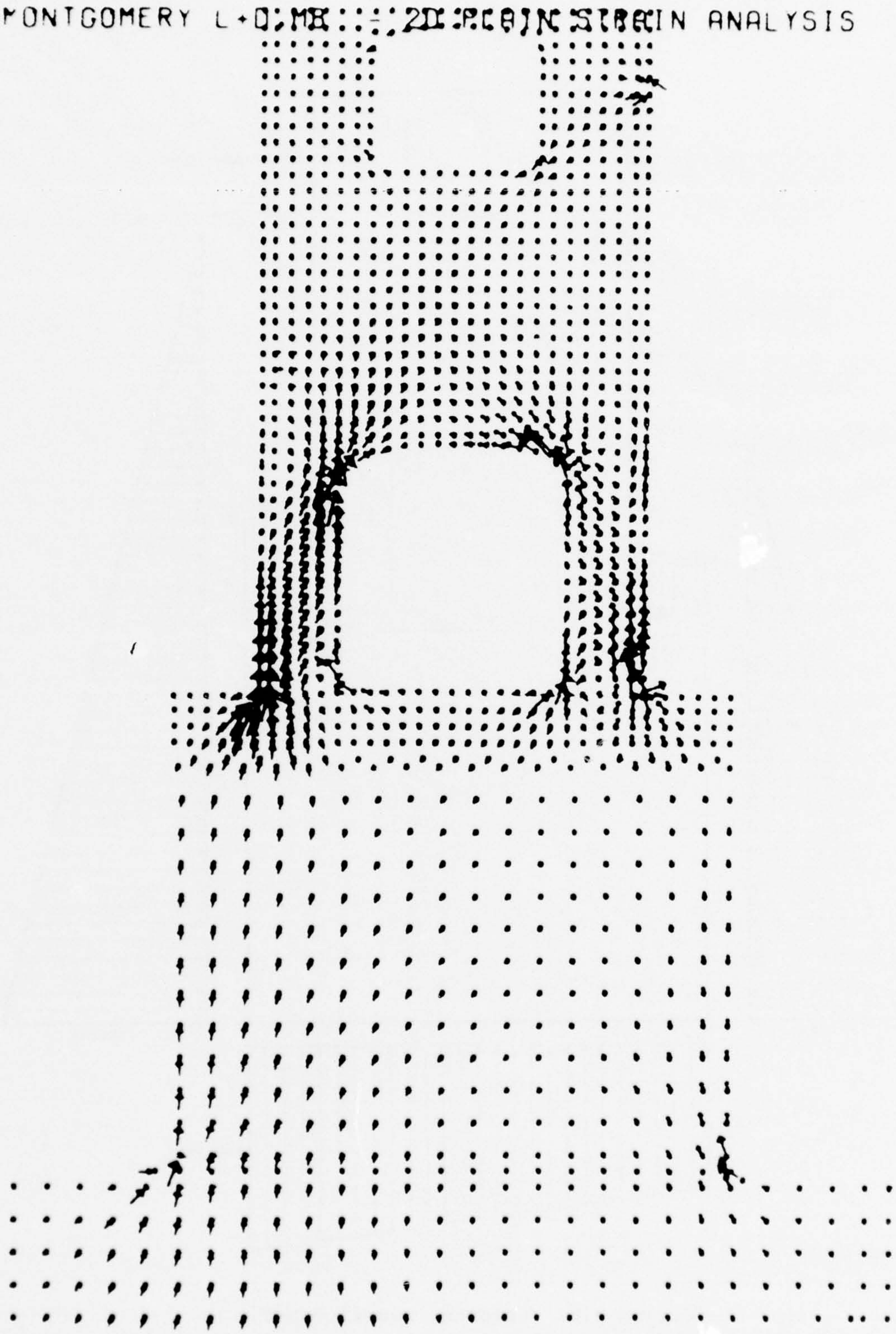


Figure 6.9. Monolith M-8, total stress distribution, normal operating case.

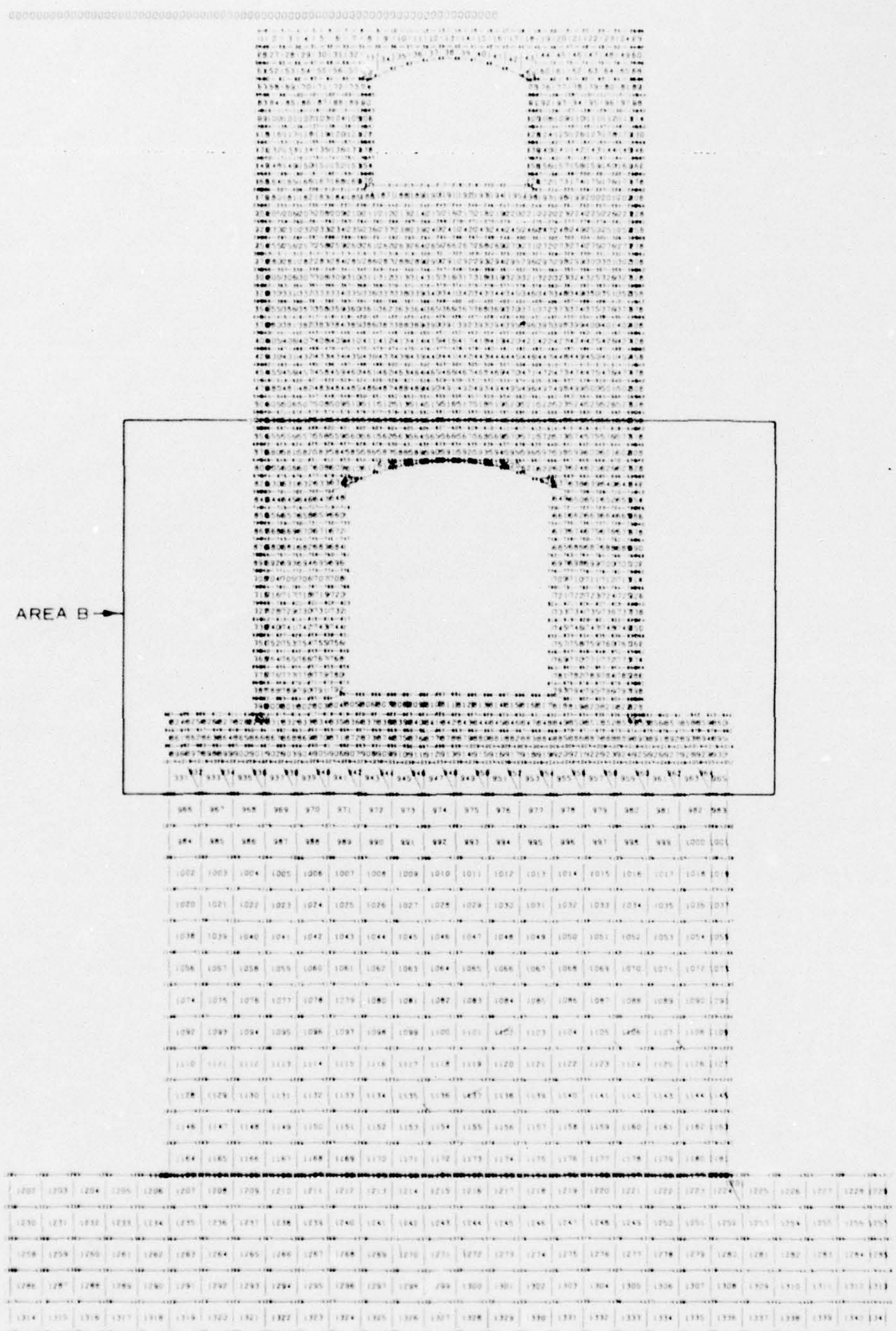
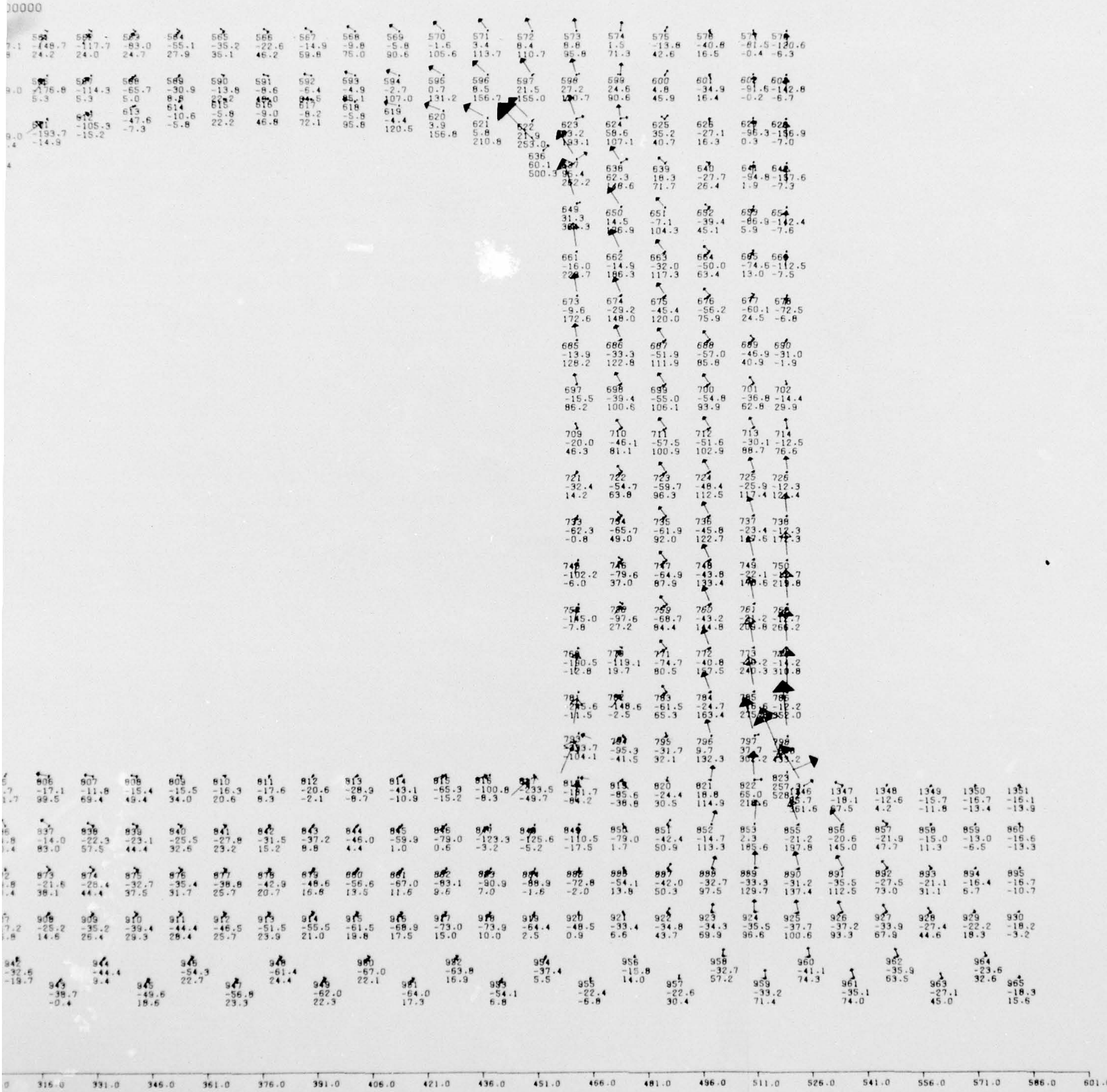


Figure 6.10. Monolith M-8 depicting areas of stress concentration ("B") for the normal operating case.

Figure 6.11. Monolith M-8, area of stress distribution as
area "B."



Monolith M-8, area of stress distribution as depicted by area "B."

2

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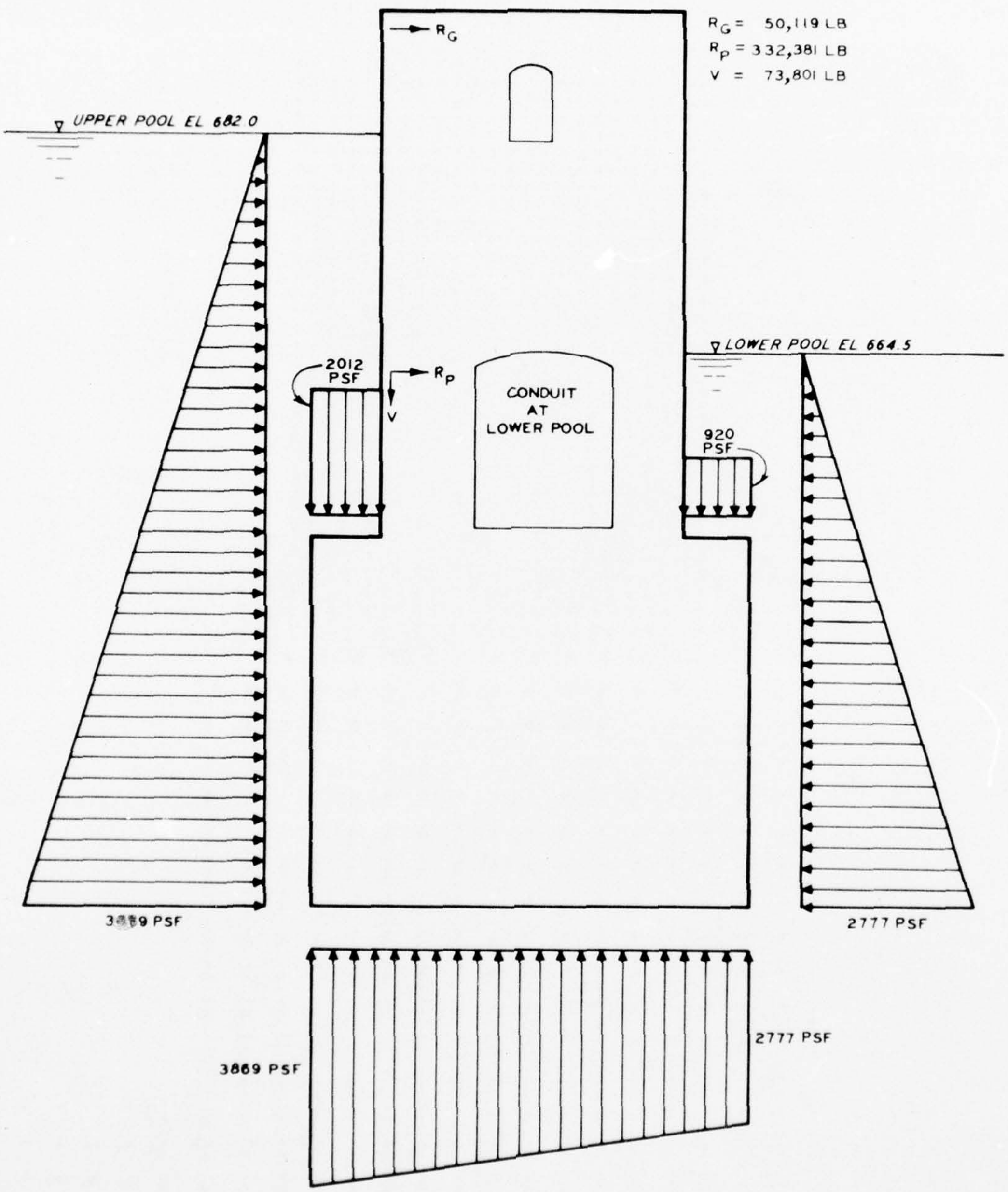


Figure 6.12. Loading, monolith M-10.

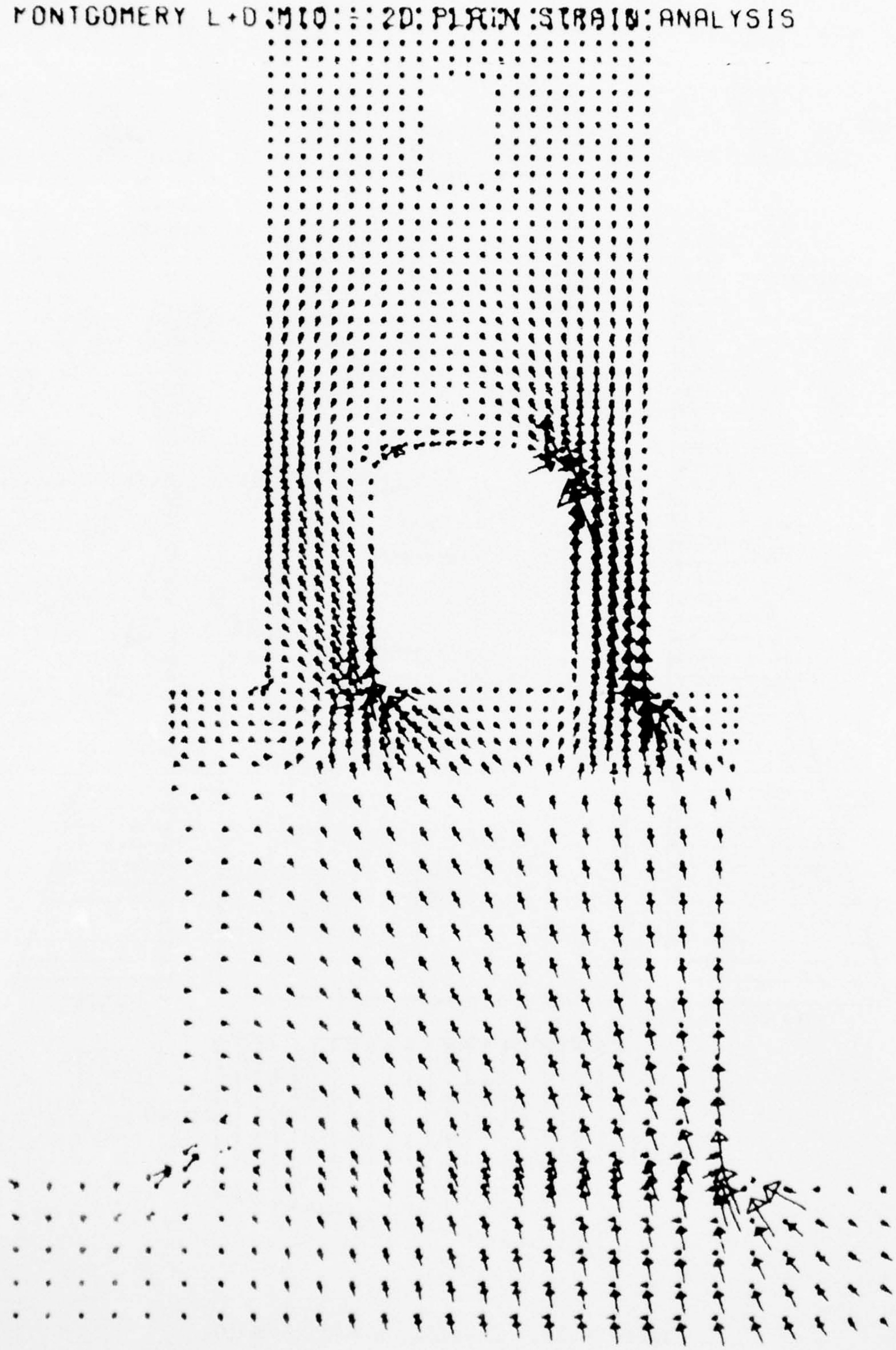


Figure 6.13. Monolith M-10, total stress distribution normal operating case.

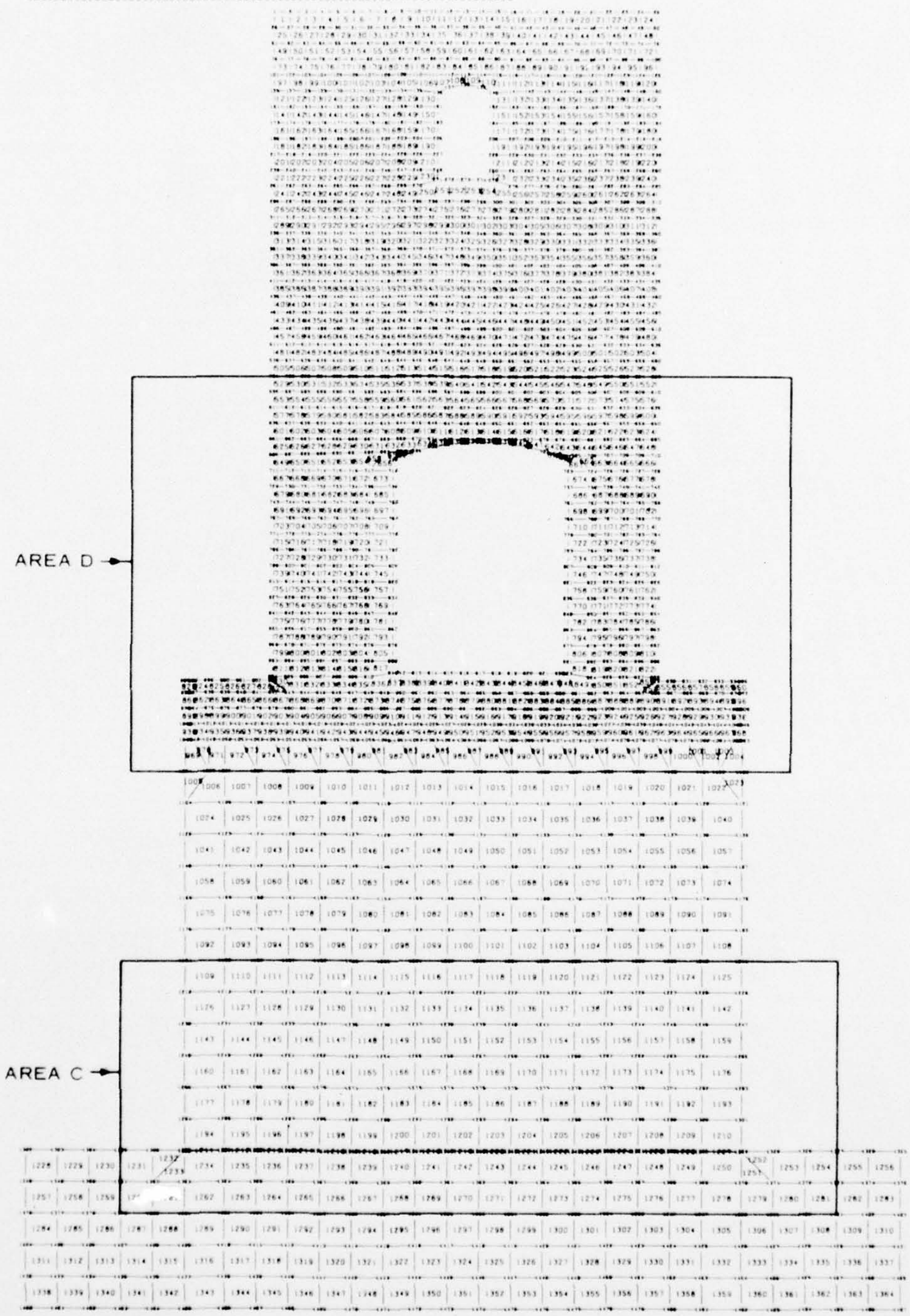
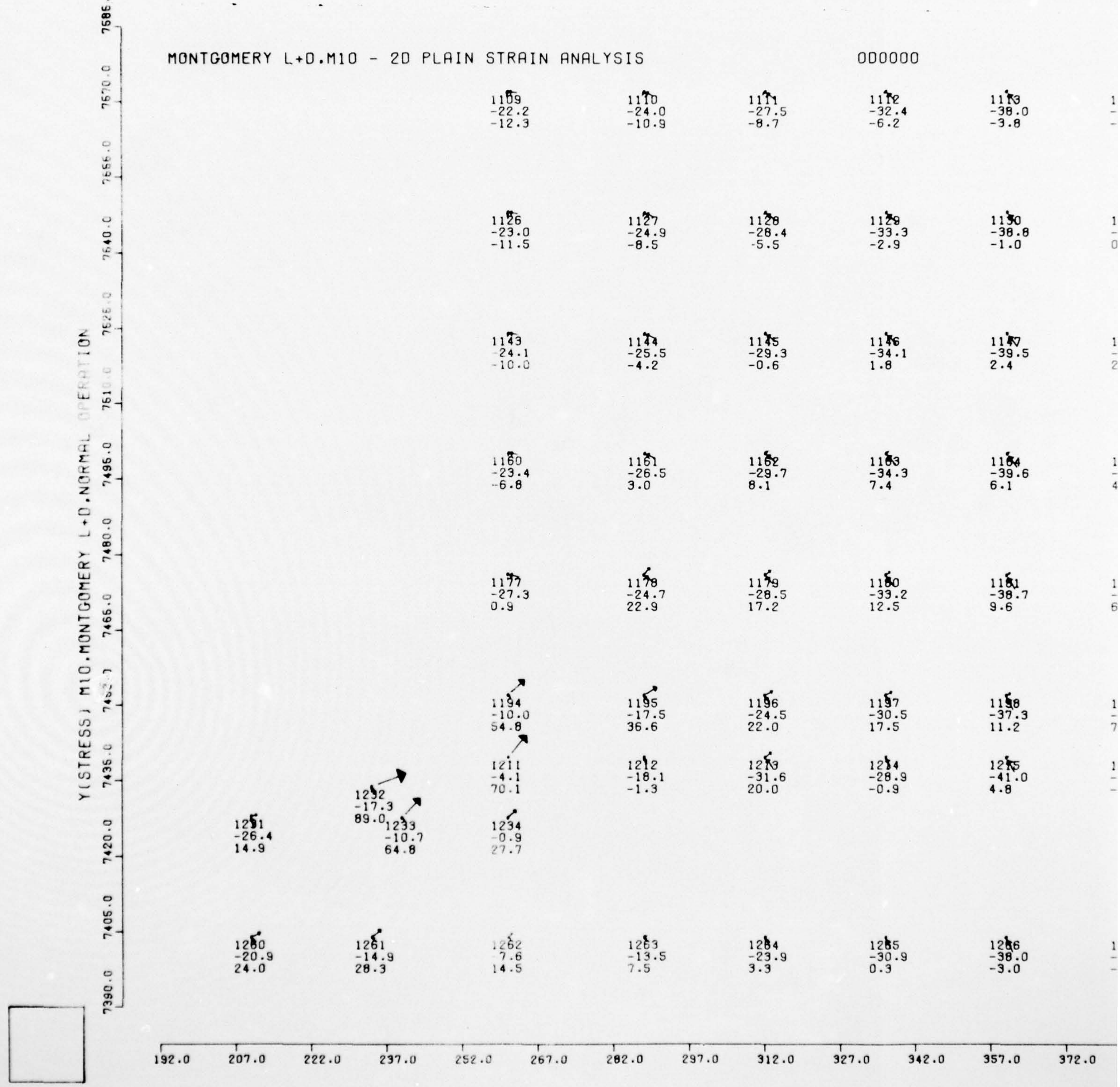


Figure 6.14. Monolith M-10 depicting areas of stress concentration ("C" and "D") for the normal operating case.


MONTGOMERY L+D.M10 - 2D PLAIN STRAIN ANALYSIS
000000
Y(STRESS) M10.MONTGOMERY L+D.NORMAL OPERATION
7585.0
7570.0
7555.0
7540.0
7525.0
7510.0
7495.0
7480.0
7465.0
7435.0
7420.0
7405.0
7390.0
1109 -22.2 -12.3
1110 -24.0 -10.9
1111 -27.5 -8.7
1112 -32.4 -6.2
1113 -38.0 -3.8
1126 -23.0 -11.5
1127 -24.9 -8.5
1128 -28.4 -5.5
1129 -33.3 -2.9
1130 -38.8 -1.0
1143 -24.1 -10.0
1144 -25.5 -4.2
1145 -29.3 -0.6
1146 -34.1 1.8
1147 -39.5 2.4
1160 -23.4 -6.8
1161 -26.5 3.0
1162 -29.7 8.1
1163 -34.3 7.4
1164 -39.6 6.1
1177 -27.3 0.9
1178 -24.7 22.9
1179 -28.5 17.2
1180 -33.2 12.5
1181 -38.7 9.6
1194 -10.0 54.8
1195 -17.5 36.6
1196 -24.5 22.0
1197 -30.5 17.5
1198 -37.3 11.2
1211 -4.1 70.1
1212 -18.1 -1.3
1213 -31.6 20.0
1214 -28.9 -0.9
1215 -41.0 4.8
1232 -17.3 89.0
1231 -26.4 14.9
1233 -10.7 64.8
1234 -0.9 27.7
1250 -20.9 24.0
1251 -14.9 28.3
1252 -7.6 14.5
1253 -13.5 7.5
1254 -23.9 3.3
1255 -30.9 0.3
1256 -38.0 -3.0
192.0
207.0
222.0
237.0
252.0
267.0
282.0
297.0
312.0
327.0
342.0
357.0
372.0

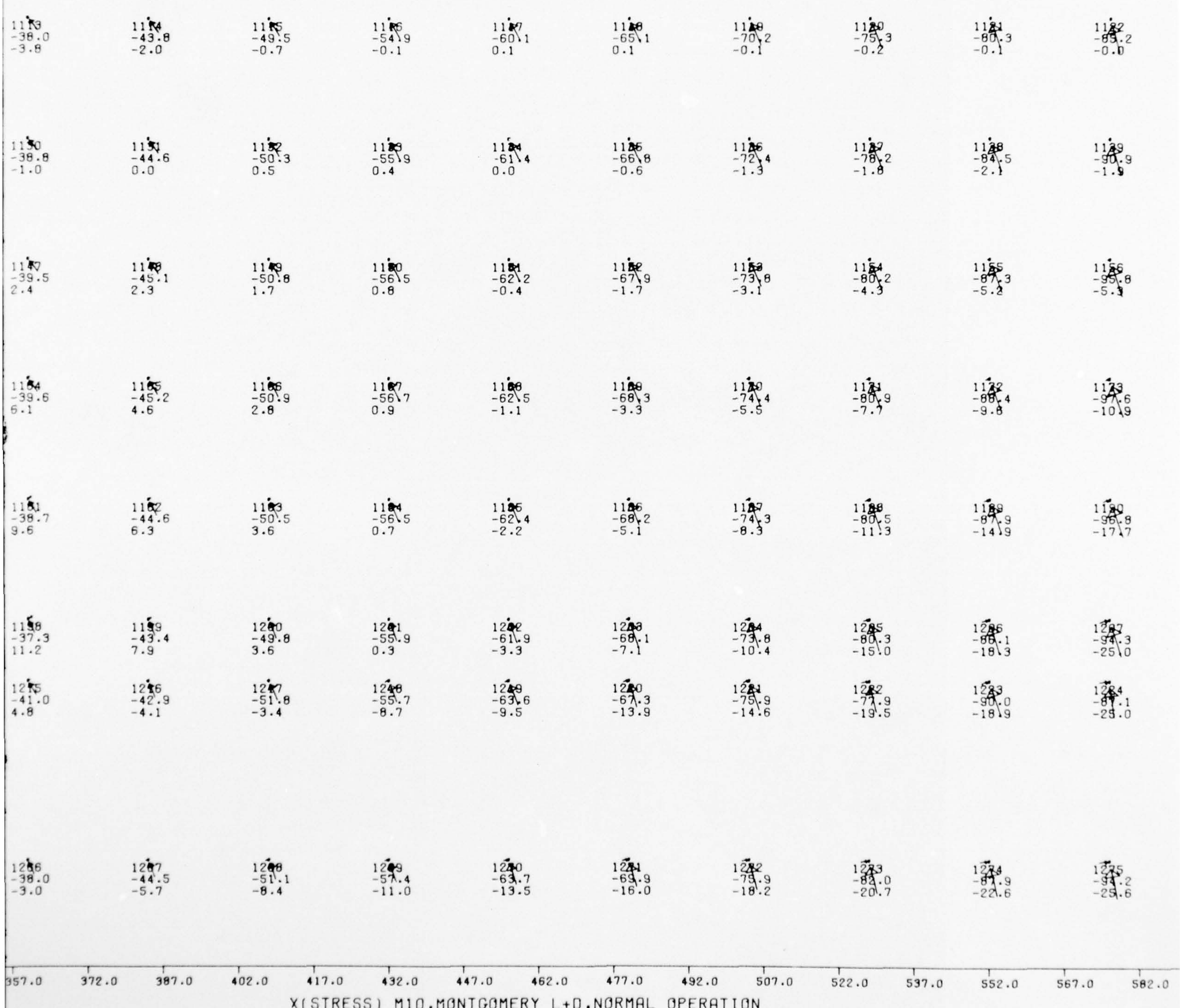


Figure 6.15. Stress concentration as depicted by area "C."

2


552.0
567.0
582.0
597.0
612.0
627.0
642.0
657.0
672.0
687.0
702.0
717.0
732.0

3

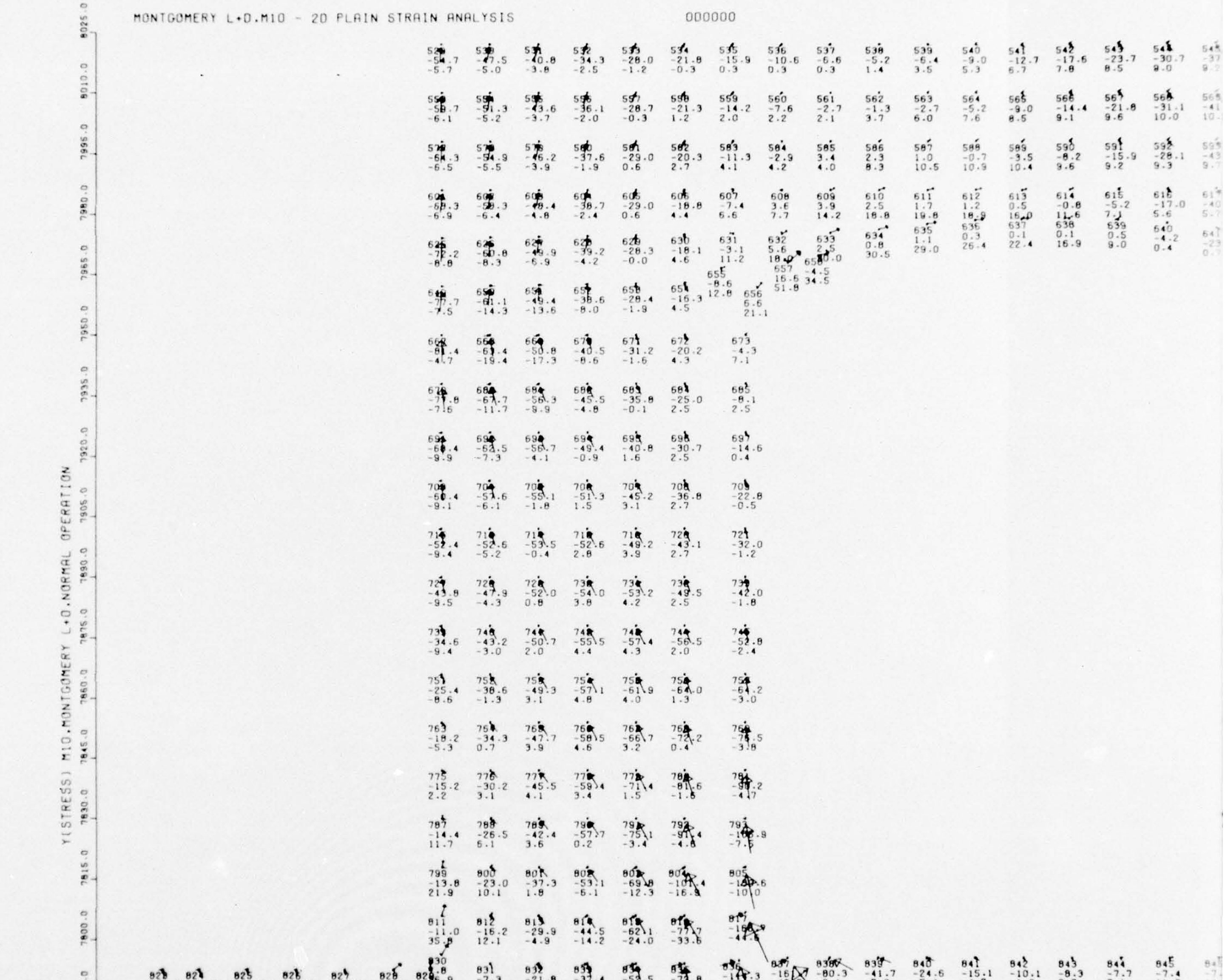
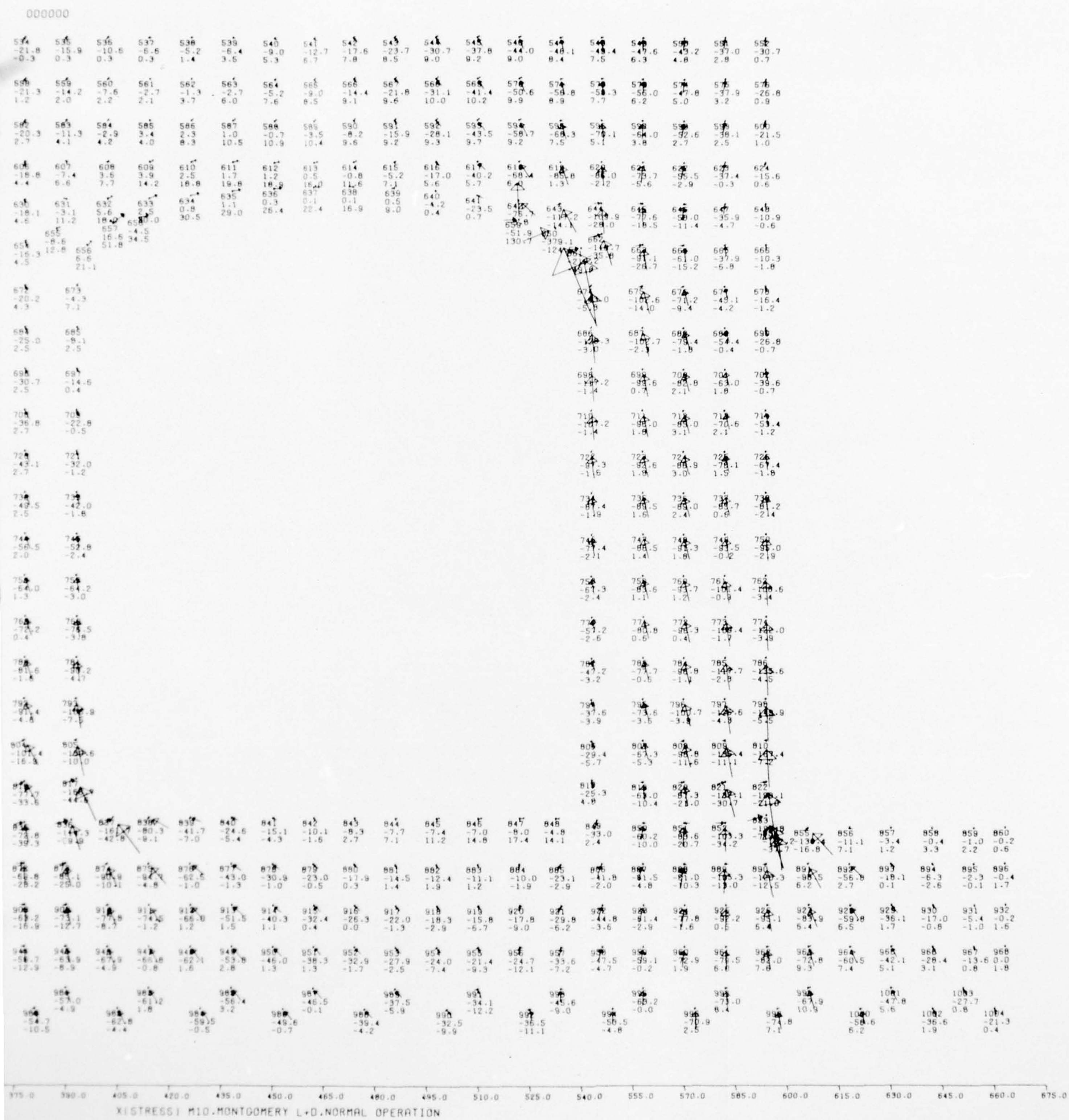


Figure 6.16. Stress concentration as depicted by a



6.16. Stress concentration as depicted by area "D."

2

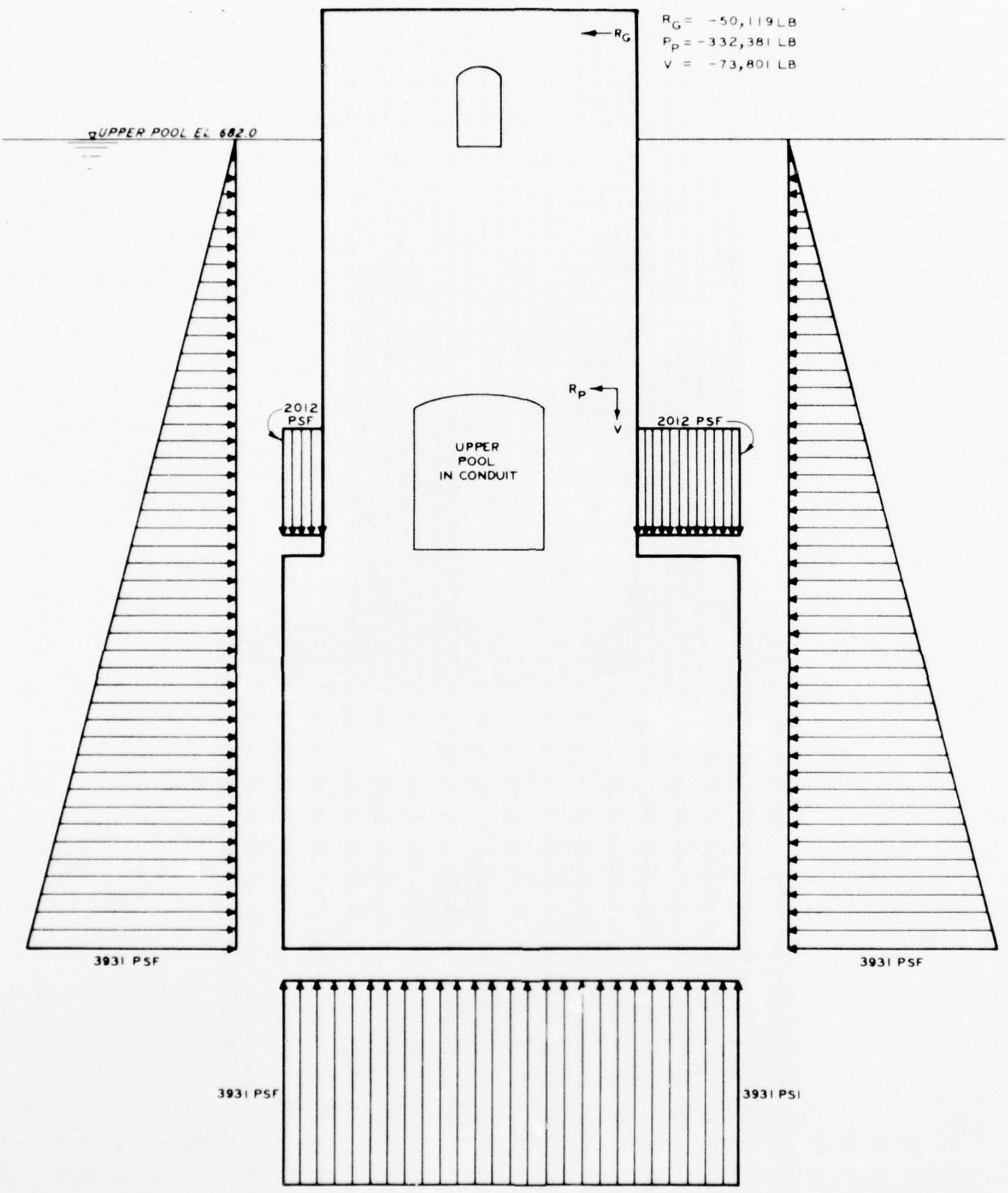


Figure 6.17. Loading, monolith R-12.

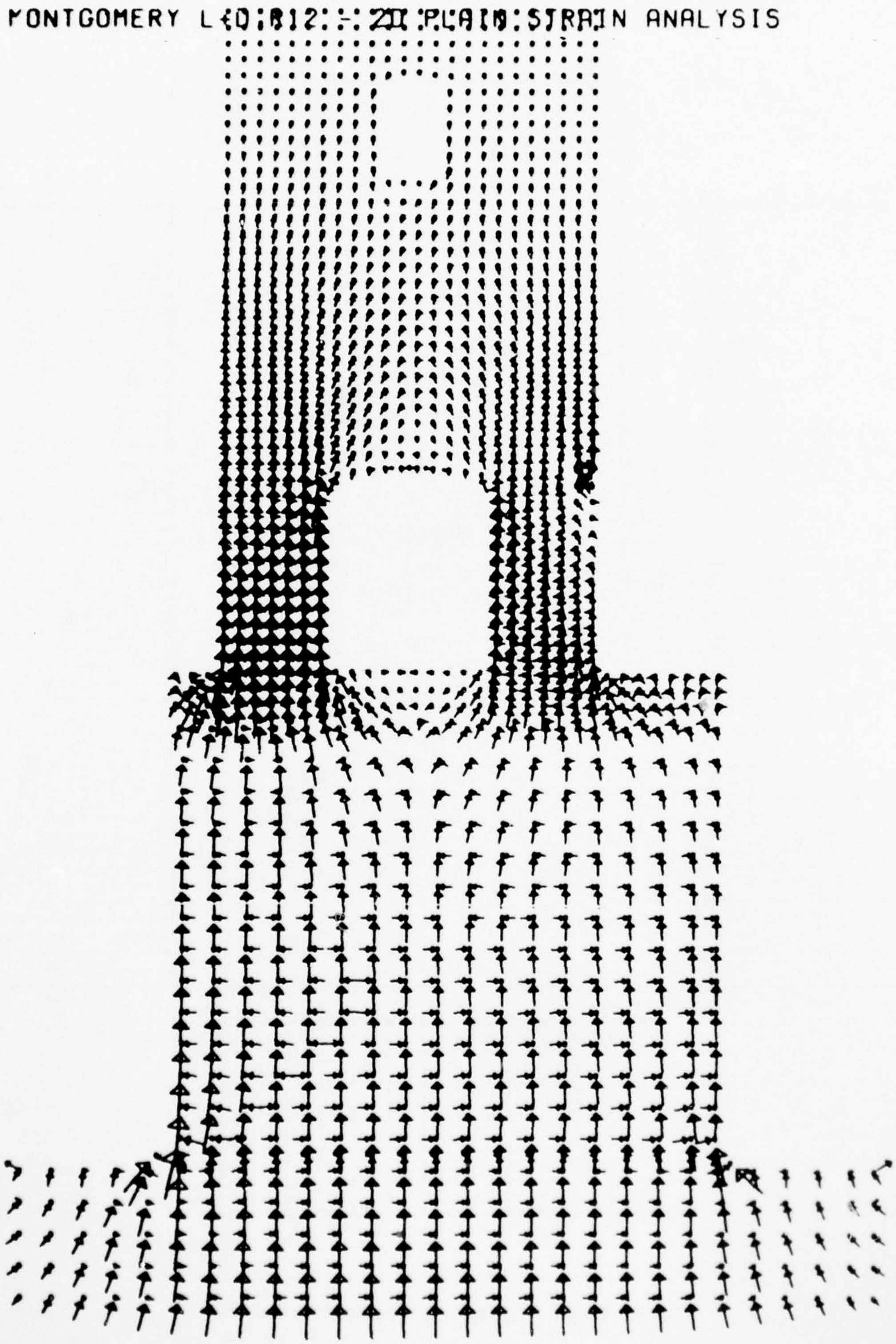


Figure 6.18. Monolith M-10, total stress distribution, normal operating case.

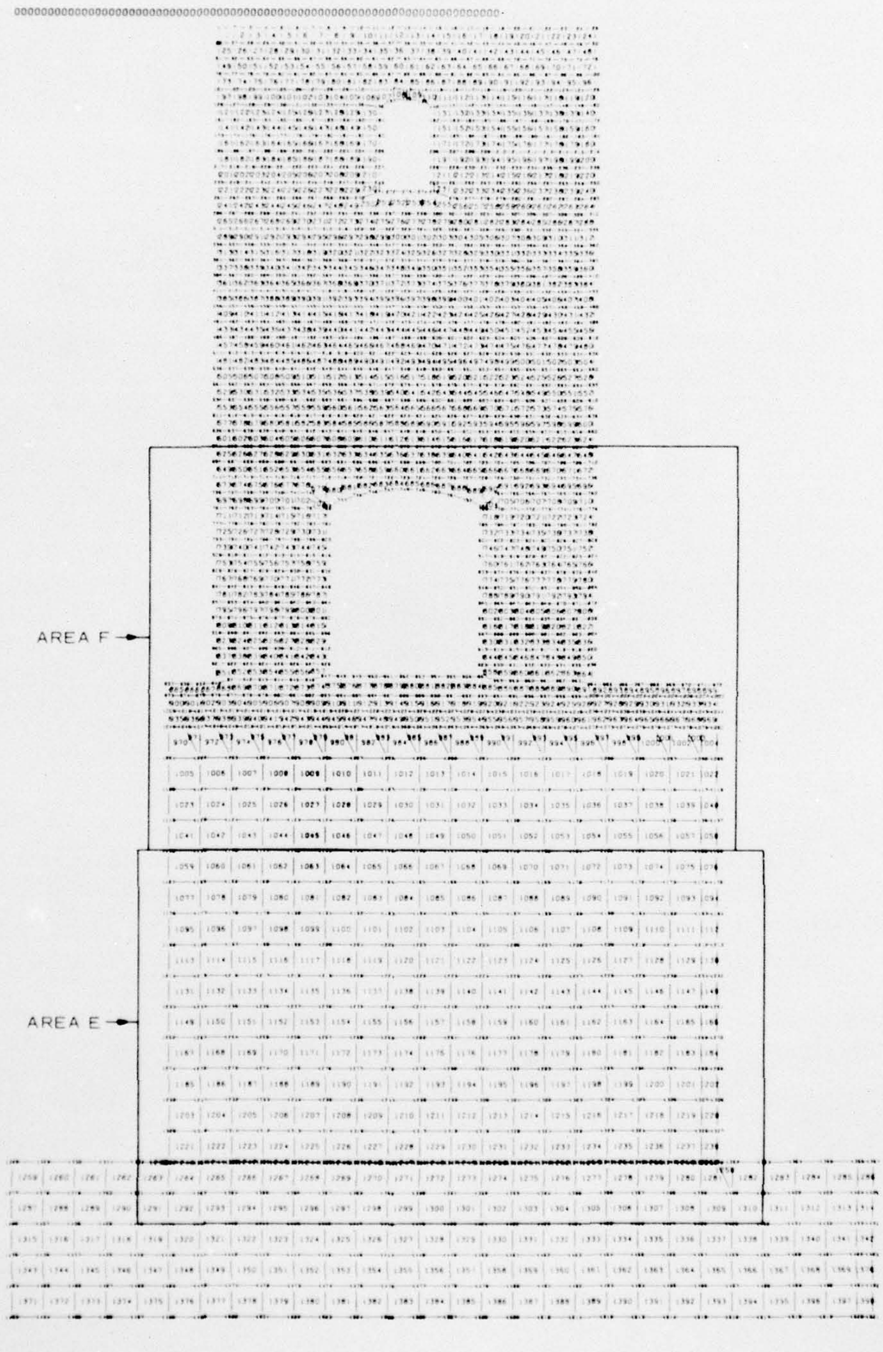


Figure 6.19. Monolith R-12 depicting area of stress concentration ("E" and "F") for the normal operating case.

Figure 6.20. Stress concentrations

AD-A038 655 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISS F/G 13/13
ENGINEERING CONDITION SURVEY AND STRUCTURAL INVESTIGATION OF MO--ETC(U)
MAR 77 C E PACE, J T PEATROSS
UNCLASSIFIED WES-MP-C-77-2 NL

2 OF 3
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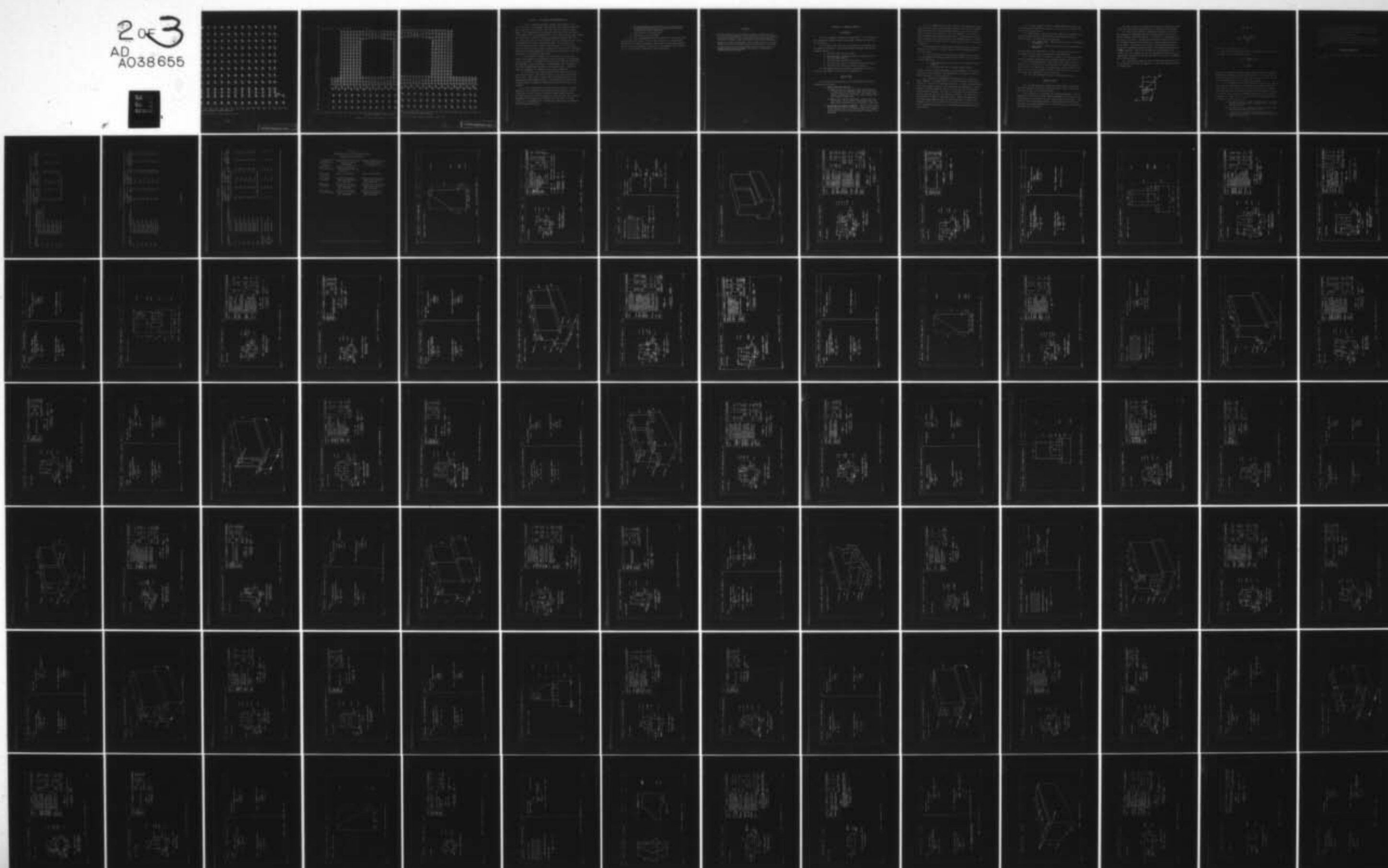
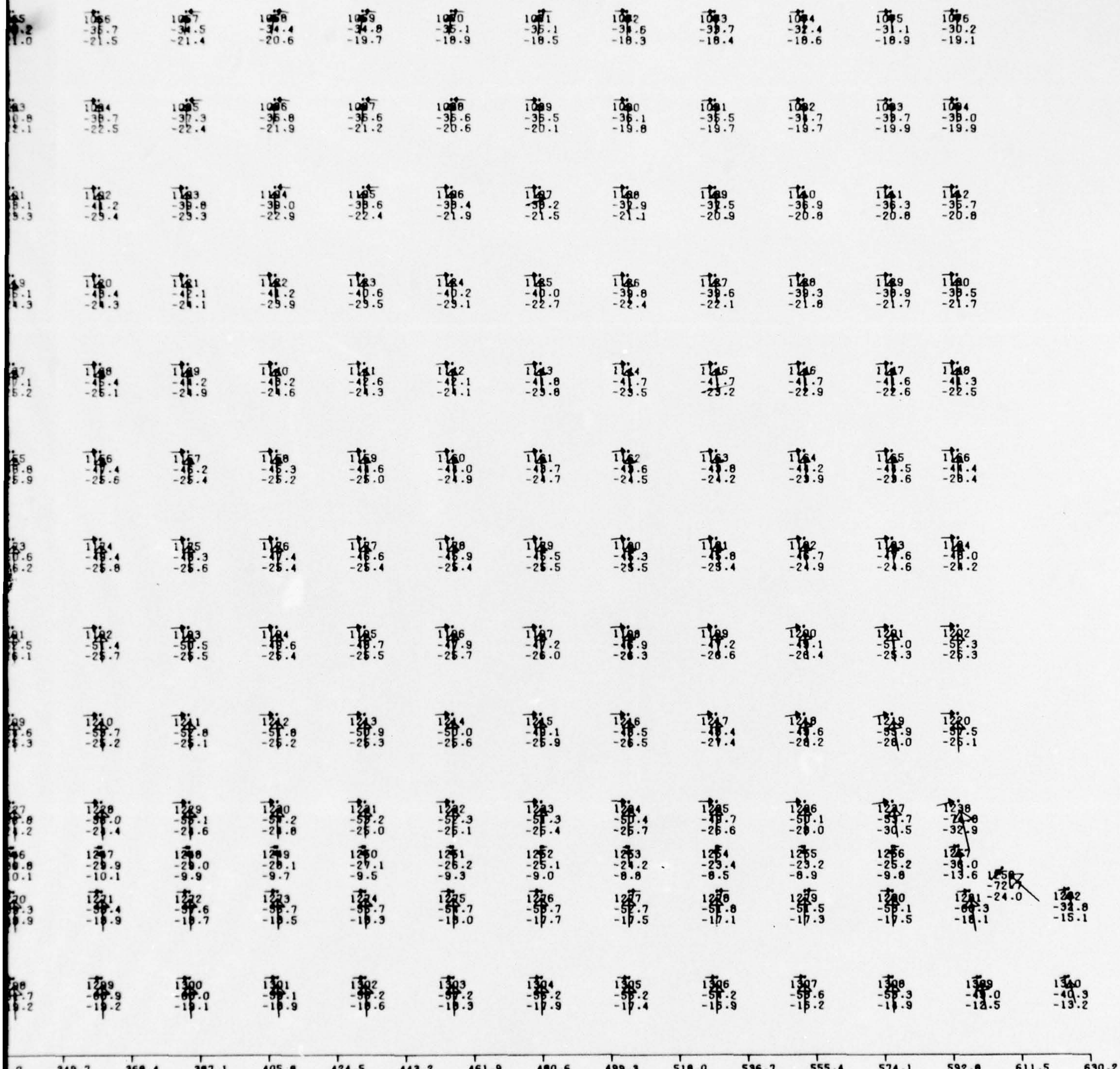




ess concentrations as depicted by area "E."

2

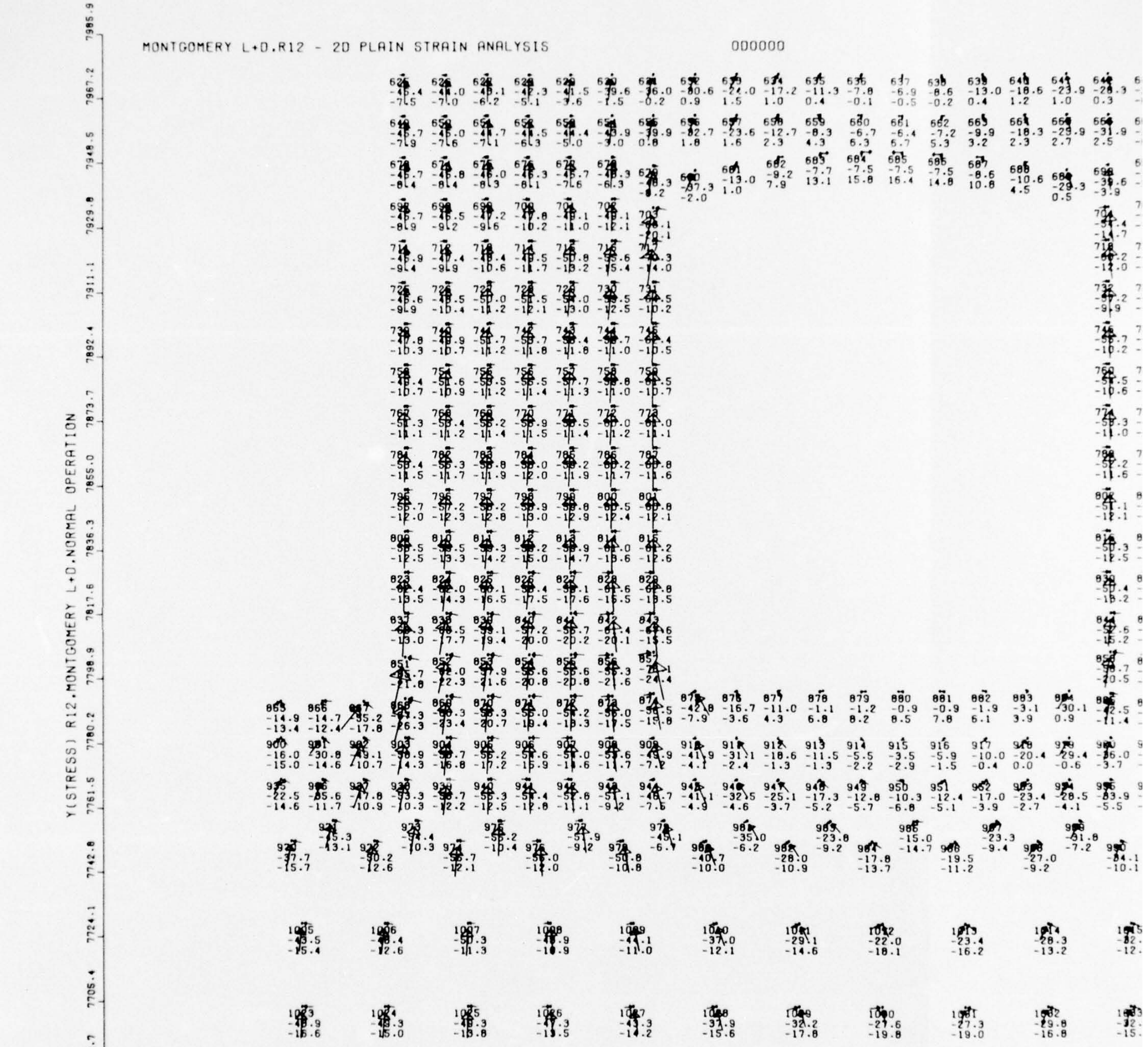
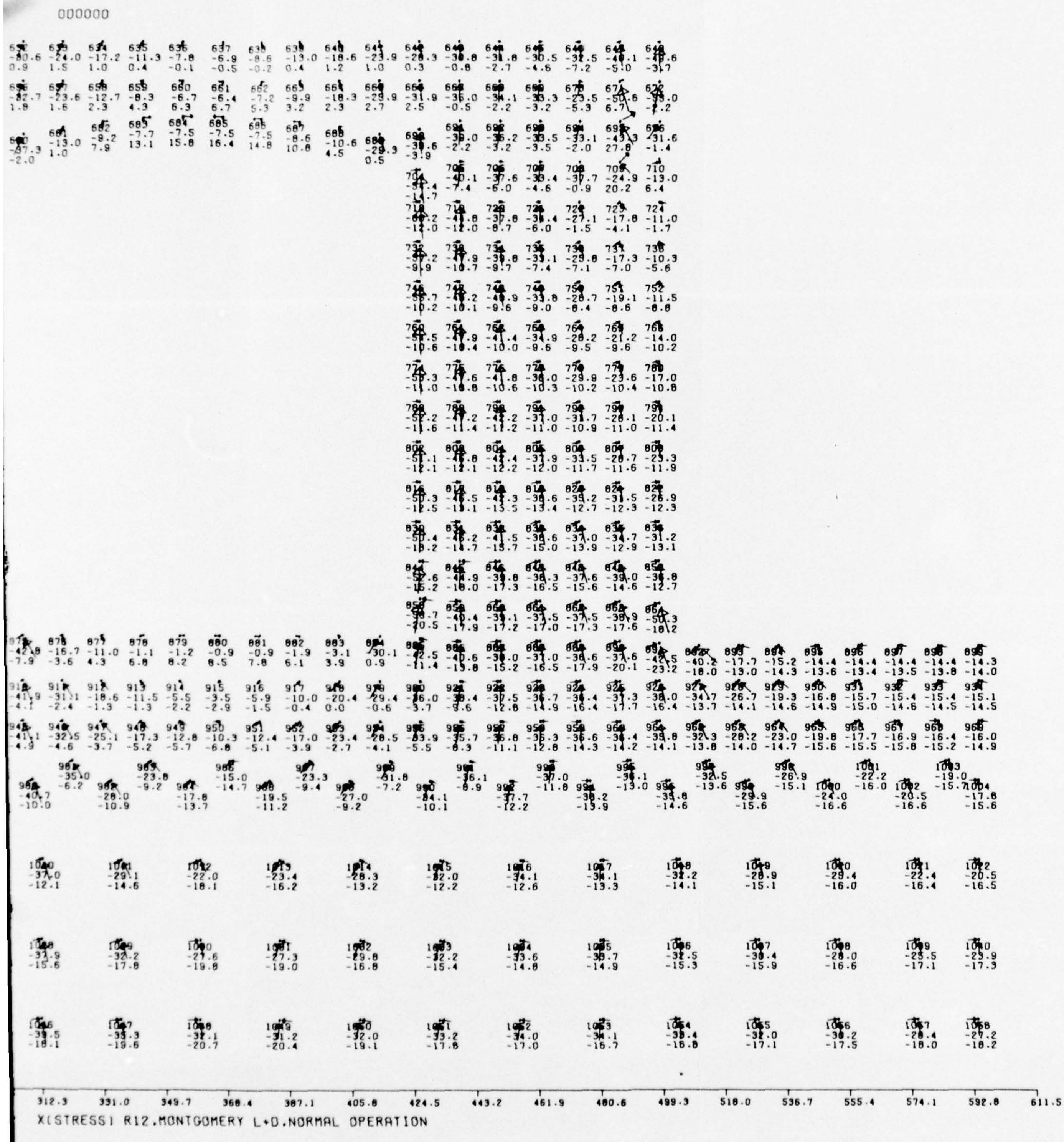


Figure 6.21. Stress concentration as depicted



.21. Stress concentration as depicted by area "F."

2

SECTION 7: CONCLUSIONS AND RECOMMENDATIONS

7.1 There is general spalling, leaching, and cracking of the concrete surfaces of Montgomery Locks and Dam. The crack survey implies that a majority of the cracking in the lock walls is caused by barge impact. The longitudinal crack parallel to the lock in the middle wall of Montgomery Lock is hypothesized to have been caused by barge impact; therefore, this can be a source of deterioration which increases with lock use. The pulse velocity study indicates that the cracking along the center of the middle wall does not worsen with depth. The concern of the cracks and spalled areas, in the concrete surface, is that they will allow the access of water; thereby causing an increased rate of deterioration due to freezing and thawing. Maintenance of the surface cracks and spalled areas is, therefore, essential.

7.2 In relation to present criteria, almost all of the monoliths on the land wall are inadequate in their resistance to overturning and base pressures. In general, the monoliths in middle and river walls are inadequate in their resistance to overturning. This is especially true for the middle-wall monoliths in the dewatered case. The miter sills are inadequate for sliding if the locks are dewatered.

7.3 The stress in the culvert wall of monolith M-8 is greater than 800 psi tension. This tensile stress is too large and will crack the concrete. This allows a stress flow up through the center of the monoliths, thereby causing cracking. This hypothesized condition for cracking should be checked by inspecting the culvert walls as soon as possible.

7.4 From the deteriorated condition of the surface of the lock monoliths, it is evident that some action must be initiated. Since corrective action is needed, a feasibility study should be made to determine what action is necessary which will provide the most economical and adequate lock usage over a period of 30 to 50 years. For this reason, it is recommended that a feasibility study be made considering the following alternatives:

a. Minimum maintenance and protection of the locks and dam from weathering with expected replacement when needed as determined by periodic inspection.

b. Rehabilitation of locks and dam.

c. Replacement of locks and dam.

7.5 The above recommendations may be affected by a total structural and operational evaluation. In fact, this study does not evaluate the foundation, steel gates, bridge work, lock gates, or appurtenant mechanical or electrical facilities. These will be considered by the Pittsburgh District in the overall evaluation of the locks and dam.

REFERENCES

1. Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures; Montgomery Locks and Dam, Ohio River, Pennsylvania, Pre-Inspection Data, First Periodic Inspection, US Army Engineer District, Pittsburgh, Corps of Engineers, Pittsburgh, Pennsylvania.

2. Condition of Piers at Montgomery Locks and Dam, Ohio River, US Army Engineer Waterways Experiment Station, Vicksburg, Miss., Apr 1974.

3. US Army Rngineer Waterways Experiment Station, Corps of Engineers, Handbook for Concrete and Cement, with quarterly supplements, Vicksburg, Miss., Aug 1949.

APPENDIX A: STABILITY ANALYSIS

Introduction

A.1 In the stability analysis, the monoliths of the locks and dam were checked for adequacy against overturning, sliding, and excessive base pressures.

A.2 In general, the stability study was done in accordance with the applicable portions of the following Engineer Manuals and Engineer Technical Letters.

a. EM 1110-2-2502, Retaining Walls.

b. EM 1110-2-2602, Planning and Design of Navigation Lock Walls and Appurtenances.

c. EM 1110-2-2607, Navigation Dam Masonry.

d. ETL 1110-2-22, Design of Navigation Lock Gravity Walls.

e. ETL 1110-2-184, Gravity Dam Design Stability.

A.3 The summary sheets and stability computations are given in Table A.1, and Figures A-1 through A-21, respectively.

Applied Loads

A.4 The lock and dam monoliths were investigated for two case loadings as given below:

a. Normal operating condition:

(1) Upper guide, land, and lower guide wall monoliths: the most critical loadings of upper pool, lower pool, and saturation level in backfill. Also, dead load, uplift, tow impact, hawser pull, wind, and gate loads were used when applicable.

(2) Middle and river wall monoliths: Normal lower and upper pools, uplift, impact, hawser pull, wind, and gate loads as applicable were considered in this case.

b. Maintenance or dewatered condition: Backfill, gate, dead loads, and uplift were considered. The saturation levels in the backfill were used as given in Table A.2. Impact, hawser pull, and wind loads were applied according to the situation.

A.5 The standard procedure was to analyze three-dimensional monoliths unless the geometry was uniform enough or could be closely approximated in order that a two-dimensional section of unit depth could be used to represent the stability of the total monolith. All sections were viewed from upstream looking downstream. Forces acting toward the right, downward, and clockwise moments are considered positive. In all cases, the lower left-hand corner of the monolith was used as the center of moments.

A.6 Approximations were necessary concerning several significant factors which affect the stability analysis; these approximations are discussed below.

A.7 The location of the resultant soil pressure was considered to be as suggested in Engineer Manual EM 1110-2-2602 for walls supported on rock foundations:

a. 0.38H above the base for horizontal or downward sloping backfill.

b. 0.45H above the base for upward sloping backfill.

It was concluded from EM 1110-2-2502, that the magnitude of horizontal soil force on the landside of the monolith can be computed by using a linear distribution of earth pressure.

A.8 In the case of Montgomery Locks and Dam, with the gravity walls supported on component rock, the "at-rest" pressure coefficient is used as the coefficient of horizontal pressure. A lower bound coefficient of at-rest pressure was used. The only way to get experimental values would be to make a number of tests at the lock and dam site using the actual backfill material. The scope of this work in time and funding was not such that this type of testing was possible. On this basis, it was decided to estimate a lower bound value. This lower bound was obtained by considering the value for sand (from dense to loosely compacted) as 0.45 to 0.55; for silt, 0.6; and for clay, from 0.7 to 1.0. It is reasonable, therefore, to use a lower bound at-rest earth coefficient of 0.5.

A.9 The unit weight of concrete, drained backfill material, and saturated backfill material was used as 151.3 lb/ft^3, 120.7 lb/ft^3, and 137.9 lb/ft^3, respectively.

A.10 Barge impact loads were applied on the basis of design loads used for locks previously constructed with considerations given in EM 1110-2-2602. The loads which were used are:

a. Lock chamber walls: 800 lb/ft but not less than 40,000 lb per monolith.

b. Other walls: 2500 lb/ft but not less than 120,000 lb per monolith.

The barge impact was considered as acting 5 ft above the waterline and was combined with the most severe normal loading conditions.

A.11 A hawser pull of 24,000 lb was applied 5 ft above pool height and was considered distributed over a monolith length of about 30 ft.

A.12 When considering gate load, hawser pull, impact loads, etc., which act on a localized area of the monolith, the loads were distributed on a per foot basis when a two-dimensional stability analysis was made. This is accurate enough for stability analysis but is not accurate enough when considering localized stresses.

A.13 Ice loads would make some case loadings more critical.

Design Criteria

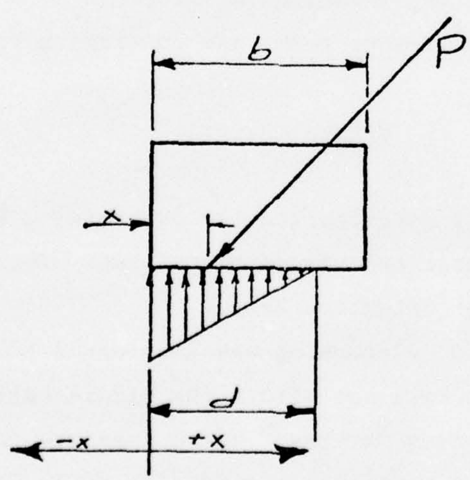
A.14 The monoliths were checked for overturning by considering where the resultant intersected the monolith base. These results were expressed as a percent of effective base.

A.15 Resistance to overturning was considered adequate if the resultant fell outside the kern but within the middle half of the base for normal operation cases using "at-rest" earth pressure coefficients. The resultant for the extreme maintenance condition using "at-rest" earth pressures was considered adequate if it fell outside the kern but within the middle half of the base.

A.16 The criteria for determining resistance to sliding are given in ETL 1110-2-184 and the safety factors are listed in ETL 1110-2-22.

A.17 There are no problems in engineering concepts if the total base pressure is compressive because for massive-rigid structures it can be obtained rather accurately by $f = P/A \pm Mc/I$ considering the total area of the base. The problem arises when the monolith just rests on a foundation and part of the base is in tension, which in reality cannot exist. If the total base is used in the analysis when part of the area is noneffective (shows tension), the equilibrium equations are not even satisfied. The way to determine the base pressures is to consider only the effective part of the monolith base--that which is in compression. This will be done and the effective area for a rectanglar base is derived below.

A.18 Consider the resultant force "x" distance from the left toe of the monolith and solve the equation $f = P/A - Mc/I$ when the stress (f) equals zero.



$$\frac{P_y}{A} - \frac{Mc}{I} = 0$$

$$\frac{P_y}{d} - \frac{\frac{d}{2} - x\, P_y \frac{d}{2}}{\frac{d^3}{12}} = 0$$

solving $d = 3x$ valid for $b > d > 0$.

A.19 The above derivation is for a two-dimensional section with a unit depth of 1 ft. The stress is then:

$$P_y(x) = \frac{f(3x)}{2} \; \frac{1}{3}(3x)$$

$$f = \frac{2}{3}\frac{P_y}{x}$$

If the resultant falls outside the base, the monolith should begin to overturn. By conventional design, the resultant falls outside the base for some of the lock monoliths. This is, in reality, not the case because the monoliths are in relatively good alignment.

A.20 In as many years as the lock has been in operation, the monoliths have not shown excessive settlement or misalignment; therefore, the resultant of all forces acting on them must fall within the base. This means that the conventional analysis is not considering some factor or factors. These factors are probably ones which are not dependable enough at this point of study to be justified in good engineering design. For example, such factors could be:

a. The force required to shear a failure wedge from behind the monolith as would have to happen for tilting of the monolith to begin.

b. The degree of uplift, which we are using in the design, may be greater then the actual situation.

c. A refinement in parameters and calculation methods is needed to more accurately obtain a horizontal soil force against the monoliths.

A.21 There are no criteria for calculating pressures when the resultant falls outside the base; all the pressure would be on the toe of the monolith giving large pressures; therefore, a value of "∞" is given for these base pressures in Table A.1.

A.22 The above is supplemental information for stability considerations and makes no analyses or conclusions concerning the monoliths of Montgomery Locks and Dam. The analyses and conclusions are given in Section 5.

Stability Computations

A.23 Stability computations are shown in the following tables and figures.

Table A.1

Summary of Stability Analysis Results

Monolith	Cases Considered	Percent Effective Base		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
L-5	Normal operation with impact Normal operation with hawser	Pile loads are below allowables					
L-17	Normal operation with gate load	75	11	4	1.65	20	120
	Maintenance	75	0	2-2/3	1.09	20	∞
L-19	Normal operation	75	16	4	1.66	20	81
	Maintenance	75	0	2-2/3	1.31	20	∞
L-25	Normal operation	75	16	4	1.66	20	80
	Maintenance	75	2	2-2/3	1.30	20	765
L-33	Normal operation	75	0	4	1.47	20	∞
	Maintenance	75	0	2-2/3	1.07	20	∞
L-42	Normal operation	Pile loads are above allowables					

(Continued)

Table A.1 (Continued)

Monolith	Cases Considered	Percent Effective Base		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
M-5	Normal operation	100	100	4	9.03	20	8.7
	Maintenance	75	47	2-2/3	1.24	20	27.4
M-7	Normal operation	100	76	4	2.67	20	13.7
	Maintenance	75	44	2-2/3	1.12	20	28.0
M-10	Normal operation	100	100	4	3.68	20	11.2
	Maintenance	75	58	2-2/3	1.24	20	22.7
M-13	Normal operation	100	75	4	1.83	20	13.7
	Maintenance	75	45	2-2/3	1.06	20	28.6
M-20	Normal operation	100	100	4	2.36	20	9.4
	Maintenance	75	77	2-2/3	1.21	20	16.5
M-22	Normal operation	100	71	4	1.89	20	15.9
	Maintenance	75	100	2-2/3	2.44	20	11.7

(Continued)

Table A.1 (Concluded)

Monolith	Cases Considered	Percent Effective Base		Sliding Safety Factor		Foundation Pressure, k/sf	
		Minimum Allowable	Actual	Minimum Allowable	Actual	Allowable	Actual
R-6	Normal operation	Pile loads are below allowables					
R-12	Normal operation	100	100	4	11.10	20	5.7
	Maintenance	75	55	2-2/3	1.10	20	23.5
R-13	Normal operation	100	82	4	2.96	20	13.8
	Maintenance	75	48	2-2/3	1.30	20	28.3
R-15	Normal operation	100	88	4	2.61	20	11.2
	Maintenance	75	47	2-2/3	1.13	20	25.3
R-20	Normal operation	100	71	4	1.33	20	15.2
	Maintenance	75	100	2-2/3	3.06	20	12.8
R-23	Normal operation	100	90	4	1.55	20	12.6
	Maintenance	75	92	2-2/3	2.90	20	15.0
R-29	Normal operation	Pile loads are below allowables					
Lower miter Sill (110' lock)	Normal operation	100	100	4	2.71	20	3.8
	Maintenance	75	100	2-2/3	6.82	20	7.8
Lower miter Sill (56' lock)	Normal operation	100	99	4	1.97	20	5.0
	Maintenance	75	100	2-2/3	5.95	20	5.5

Table A.2

Saturation Levels to Use in the Backfill

of the Land Wall Monoliths

Sections of Land Side Lock Wall	Saturation Elevations for Normal Operating Conditions	Saturation Elevations for Extreme Maintenance Conditions
Upper guide wall monolith	One-half way between upper pool and the top of lock wall	--
Upper gate monoliths	Upper pool elevation	Upper pool elevation
Lock chamber monoliths	One-half way between upper pool and lower pool elevations	Three-fourths way between upper pool and lower pool elevations
Lower gate monoliths	One-half way between upper pool and lower pool elevations	Three-fourths way between upper pool and lower pool elevations
Lower guide wall monoliths	One-half way between upper pool and lower pool elevations	One-half way between upper pool and lower pool elevations

SUBJECT: LAND WALL - UPPER GUARDWALL L-5

COMPUTED BY: DATE:

CHECKED BY: DATE:

STATION 8+95.0A TO 8+50.0A

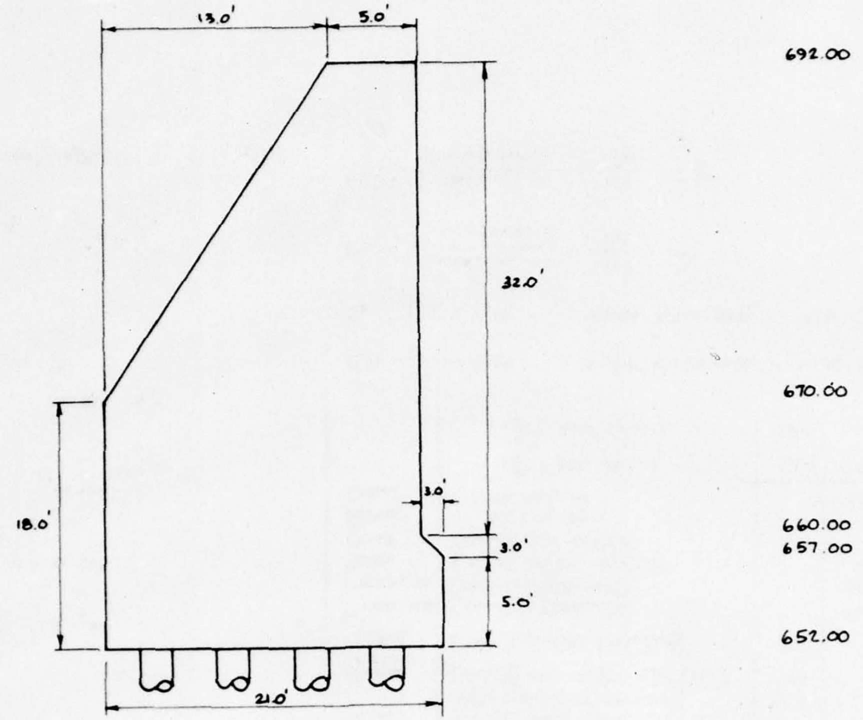


FIGURE A-1 LAND WALL - UPPER GUARD WALL MONOLITH L-5

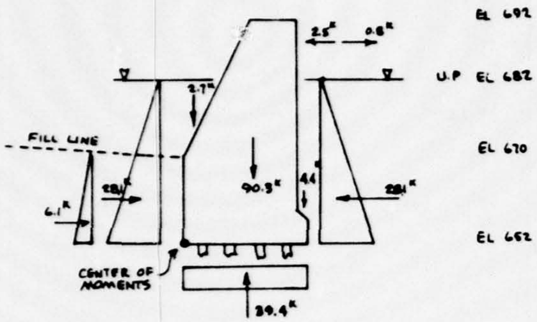
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - UPPER WALL L-5

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM		F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513]((5)(40) + (1/2)(3)(3) + (3)(5) + (1/2)(13)(22) + (13)(18))	90.3		10.46	945
W_{WATER} RIVER SIDE	[.0625][(682 - 660)(3) + (1/2)(3)(3)]	4.4		19.53	86
W_{WATER} LAND SIDE	[.0625]((1/2)(682 - 670)(7.09))	2.7		2.36	6
$P_{WATER, RIVER}$	$[.0625](1/2)(682-652)^2$		-28.1	10.00	-281
$P_{WATER, LAND}$	$[.0625](1/2)(682-652)^2$		28.1	10.00	281
P_{EARTH}	[.0754](1/2)(670-652)(.5)		6.1	7.00	43
UPLIFT	[.0625](682-652)(21)	-39.4		10.50	-414
HAWSER	0.8 KIPS/FT		0.8	35.00	28
IMPACT	2.5 KIPS/FT		-2.5	35.00	-87
	(1) WITH IMPACT	58.0	+3.6		579
	(2) WITH HAWSER	58.0	+6.9		694

$e_1 = \frac{579}{58.0} = 9.98$ PERCENT ACTIVE BASE = 100% (COMPRESSION)

$e_2 = \frac{694}{58.0} = 11.96$ PERCENT ACTIVE BASE = 100% (COMPRESSION)

$F.S._1 = \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{1377}{738} = 1.87$

$F.S._2 = \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{1290}{766} = 1.68$

NORMAL OPERATIONS

UPPER POOL IN RIVER
FILL ELEV (AT MONOLITH FACE) 670.00

FIGURE A-1 LAND WALL - UPPER WALL MONOLITH L-5

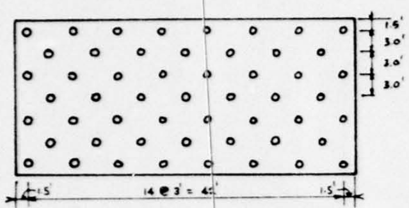
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - UPPER WALL L-5

COMPUTED BY | DATE

CHECKED BY | DATE

HORIZONTAL PILE LOADS



ALLOWABLE HORIZONTAL LOAD PER PILE = 8^K

NORMAL OPERATIONS

CASE I WITH IMPACT $F_H = \frac{3.6(45)}{53} = 3.06\ ^K/\text{pile} < 8^K$

CASE II WITH HAWSER $F_H = \frac{6.9(45)}{53} = 5.86\ ^K/\text{pile} < 8^K$

BASE PILE PRESSURE

NORMAL OPERATIONS $\quad I_{\text{PILE GROUP}} = 1529.4\ FT^4$

CASE I WITH IMPACT

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{45(58)}{.7854(53)} + \frac{45(58)(10.5-10)(9)}{1529.4}$$
$$= 62.7 + 7.68$$
$$= 70.38\ KSF$$

$$70.38\frac{K}{ft^2} \times .7854\frac{ft^2}{pile} = 55.3\frac{K}{pile}$$
$$\text{ALLOWABLE} = 100\frac{K}{pile}$$

CASE II WITH HAWSER

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{45(58)}{.7854(53)} + \frac{45(58)(11.98-10.5)(9)}{1529.4}$$
$$= 62.7 + 22.73$$
$$= 85.43\ KSI$$

$$85.43\frac{K}{ft^2} \times .7854\frac{ft^2}{pile} = 68.1\frac{K}{pile} < 100\frac{K}{pile} \quad OK$$

FIGURE A-1 LAND WALL - UPPER G WALL MONOLITH L-5

WES FORM NO. 1253A DECEMBER 1964

SUBJECT	COMPUTED BY	DATE
LAND WALL - UPPER GATE MONOLITH L-17	CHECKED BY	DATE

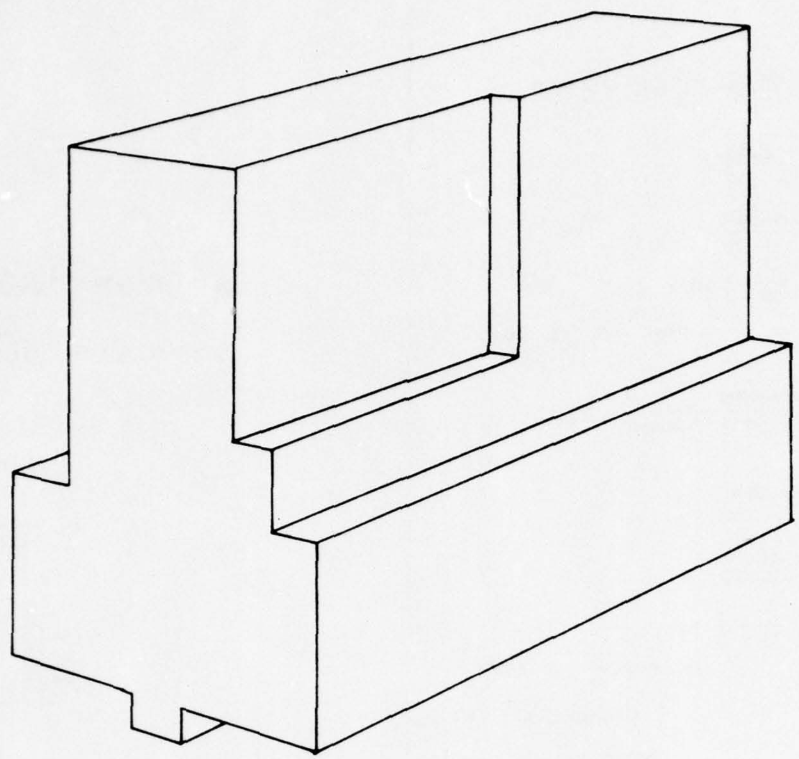


FIGURE A-2 LAND WALL - UPPER GATE MONOLITH L-17

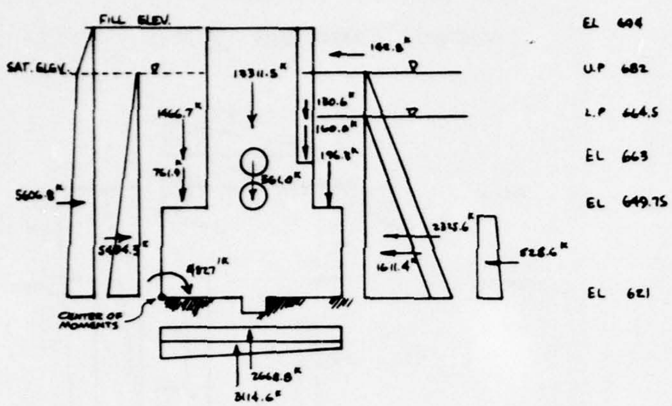
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - UPPER GATE MONOLITH L-17

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE I NORMAL OPERATIONS

UPPER POOL IN LOCK
LOWER POOL ABOVE LOCK

ITEM	FACTOR	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513][(24)(44.25)(47.25) + (47.25)(29.25)(35) + (5)(8)(47.25) - (5.5)(20)(31) - (2)(π)(5.5)²(47.25) - (3)(47.25)(.5)	13311.5		18.48	245997
$W_{WATER, LOCK}$	[.0625](682 - 663)(5.5)(20)	130.6		29.25	3820
$W_{WATER, LOCK}$	[.0625][(682 - 649.75)(3)(20) + (664.5 - 649.75)(3)(27.25)]	196.3		33.50	6576
$W_{WATER, LAND}$	[.0625](682 - 649.75)(8)(47.25)	761.9		4.00	3048
$W_{WATER, CULVERT}$	[.0625](2)(π)(5.5)²(47.25)	561.0		17.25	9677
W_{EARTH}	[.1207](694 - 682)(8)(47.25) + [.0754](682 - 649.75)(8)(47.25)	1466.7		4.00	5866
$P_{WATER, LOCK}$ (IN LOCK)	[.0625](½)(664.5 - 621)²(27.25)		-1611.4	14.50	-23365
(ABOVE LOCK)	[.0625](½)(682 - 621)²(20.0)		-2325.6	20.33	-47279
$P_{WATER, LAND}$	[.0625](½)(682 - 621)²(47.25)		5494.3	20.33	111699
P_{EARTH}	[.1207][(½)(694 - 682)²(.5) + (694 - 682)(.5)(682 - 621)] + [.0754](½)(682 - 621)²(.5)		5606.8	27.74	155533
P_{EARTH} & CONC IN LOCK	[.1513](649.75 - 647.75)(.5)(647.75 - 621)(47.25) + [.0754](647.75 - 621)²(.5)(½)(47.25)		-828.6	9.94	-8236
UPLIFT (ABOVE, LOCK)	[.0625](682 - 621)(35)(20)	-2668.8		17.50	-46704
(IN LOCK)	[.0625][(664.5 - 621)(35)(27.25) + (½)(682 - 664.5)(35)(27.25)]	-3114.6		16.52	-51465
GATE S			-142.8	71.67	-10234
GATE V		160.0		29.33	4693
GATE M					4827
		10804.6	6192.7		364453

$$e = \frac{364453}{10804.6} = 33.73$$

$$\text{PERCENT ACTIVE BASE} = \frac{(35 - 33.73)(3)(100)}{35} = 10.88\%$$

$$F.S. = \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{390017}{376321} \cong 1.04$$

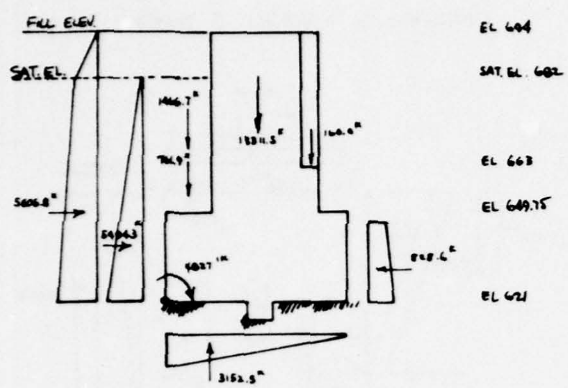
FIGURE A-2 LAND WALL - UPPER GATE MONOLITH L-17

SUBJECT: LAND WALL - UPPER GATE MONOLITH L-17

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE II MAINTENANCE CONDITION

SATURATION ELEVATION 682
LOCK DEWATERED

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		13311.5		18.48	245997
$W_{WATER, LAND}$		761.9		4.00	3048
W_{EARTH}		1466.7		4.00	5866
$P_{WATER, LAND}$			5494.3	20.33	111699
P_{EARTH}			5606.8	27.74	155533
$P_{EARTH \,\&\, CONC.}$			-828.6	9.94	-8236
UPLIFT	[.0625] (682-621)(35)(47.25)	-3152.5		11.67	-36779
GATE V		160.0		29.33	4693
GATE M					4827
		12547.6	10272.5		486648

$$e = \frac{486648}{12547.6} = 38.78$$

Percent Active Base: Resultant falls outside of base

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{298136}{345607} = 0.86$$

FIGURE A-2 LAND WALL - UPPER GATE MONOLITH L-17

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - UPPER GATE MONOLITH L-17

COMPUTED BY: DATE:

CHECKED BY: DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \sum F_v \tan\phi + \text{key Resistance}$$
$$= (10804.6)(.5658) + (8)(47.25)(144)(.075)$$
$$= 6113.2 + 4082.4$$
$$= 10195.6$$

$$SSF = 10195.6 / 6192.7$$
$$= 1.65$$

CASE II MAINTENANCE CONDITION

$$R = (12547.6)(.5658) + 4082.4$$
$$= 7099.4 + 4082.4$$
$$= 11181.8$$

$$SSF = 11181.8 / 10272.5$$
$$= 1.09$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \left(\frac{2}{3}\right)\left(\frac{?}{e}\right) / L$$
$$= \frac{(2)(10804.6)}{(3)(25 - 33.73)} / 47.25$$
$$= 120.0 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

Base Pressures are Very large

FIGURE A-2 LAND WALL - UPPER GATE MONOLITH L-17

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL- UPPER VALVE MONOLITH L-19

COMPUTED BY:

DATE

CHECKED BY:

DATE

STATION 3+60 A TO 3+14 A

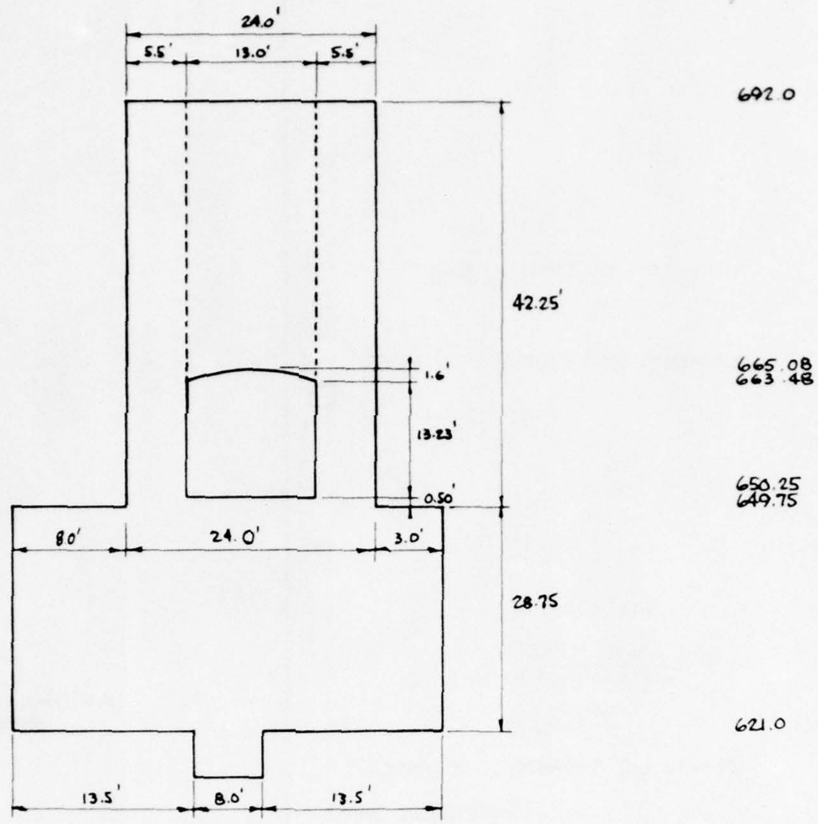
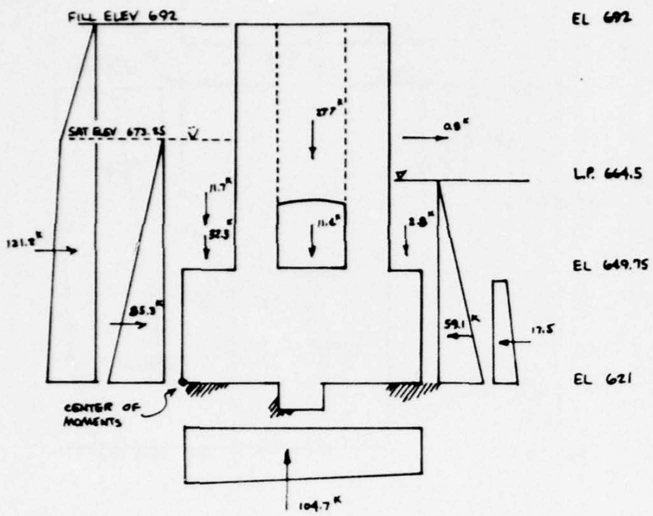


FIGURE A-3 LAND WALL- UPPER VALVE MONOLITH L-19

NOT TO SCALE



CASE I NORMAL OPERATIONS

SATURATION ELEV 673.25
LOWER POOL IN LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{conc.}$	[.1513][(24)(71) + (8)(28.75) + (3)(28.75) + (8)(5) - (13)(13.23 + .77) - (27.8)(13)(6)/46]	277.0		18.57	5144
$W_{water, lock}$	[.0625](664.5 - 649.75)(3)	2.8		33.50	94
$W_{water, land}$	[.0625](673.25 - 649.75)(8)	11.7		4.00	47
$W_{water, culvert}$	[.0625](14)(13)	11.4		20.00	228
$W_{earth, land}$	[.1207](692 - 673.25)(8) + [.0754](673.25 - 649.75)(8)	32.3		4.00	129
$P_{water, lock}$	$[.0625](\frac{1}{2})(664.5 - 621)^2$		-59.1	14.50	-857
$P_{water, land}$	$[.0625](\frac{1}{2})(673.25 - 621)^2$		85.3	17.42	1486
$P_{earth, land}$	$[.1207][(\frac{1}{2})(692 - 673.25)^2(.5) + (692 - 673.25)(.5)(673.25 - 621)] + [.0754](\frac{1}{2})(673.25 - 621)^2(.5)$		121.2	26.98	3270
$P_{earth \& conc, lock}$	$[.1513](649.75 - 647.75)(.5)(647.75 - 621) + [.0754](\frac{1}{2})(647.75 - 621)^2(.5)$		-17.5	9.94	-174
UPLIFT	$[.0625][(664.5 - 621)(35) + (673.25 - 664.5)(35)(\frac{1}{2})]$	-104.7		16.97	-1777
HAWSER	0.8 KIPS/FT		0.8	48.50	39
		230.5	130.7		7629

$$e = \frac{7629}{230.5} = 33.10'$$

$$\text{PERCENT ACTIVE BASE} = \frac{(35 - 33.1)(3)(100)}{35} = 16.29\%$$

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{7121.3}{6682.7} = 1.07$$

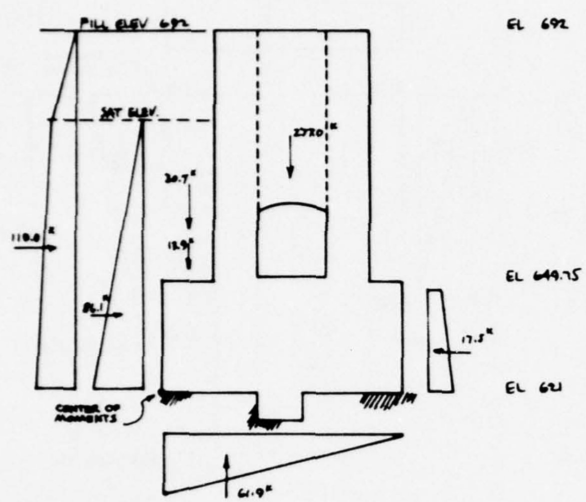
FIGURE A-3 LAND WALL - UPPER VALVE MONOLITH L-19

SUBJECT: LAND WALL - UPPER VALVE MONOLITH L-19

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$		277.0		18.57	5144
$W_{WATER, LAND}$	$[.0625](664.5 + (3/4)(682-664.5) - 649.75)(8)$	13.9		4.00	56
$W_{EARTH, LAND}$	$[.1207](692-677.62)(8) + [.0754](677.62-649.75)(8)$	30.7		4.00	123
$P_{WATER, LAND}$	$[.0625](1/2)(664.5 + (3/4)(682-664.5) - 621)^2$		86.1	21.38	1840
$P_{EARTH, LAND}$	$[.1207][(1/2)(692-677.62)^2(.5) + (692-677.62)(.5)(677.62-621)] + [.0754](677.62-621)^2(1/2)(.5)$		110.0	26.98	2968
$P_{EARTH\ \&\ CONC., LOCK}$			-17.5	9.94	-174
UPLIFT	$[.0625](1/2)(677.62-621)(35)$	-61.9		11.67	-722
		259.7	178.6		9235

$$e = \frac{9235}{259.7} = 35.56'$$

Resultant falls outside the base

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{6107.7}{6252.1} = .98$$

CASE II MAINTENANCE CONDITION

SATURATION ELEVATION 677.62
LOCK DEWATERED

FIGURE A-3 LAND WALL - UPPER VALVE MONOLITH L-19

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - UPPER VALVE MONOLITH L-19

COMPUTED BY: DATE:

CHECKED BY: DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \sum F_V \tan\phi + \text{KEY RESISTANCE}$$
$$= (230.5)(.5658) + (8)(1)(.075)(144)$$
$$= 130.4 + 86.4$$
$$= 216.8$$

$$SSF = 216.8/130.7$$
$$= 1.66$$

CASE II MAINTENANCE CONDITION

$$R = (259.7)(.5658) + 86.4$$
$$= 233.3$$

$$SSF = 233.3/178.6$$
$$= 1.31$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \frac{2}{3}\frac{P}{e}$$
$$= \frac{(2)(230.5)}{(3)(35-33.1)}$$
$$= 80.9 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

Base Pressures are very large

FIGURE A-3 LAND WALL- UPPER VALVE MONOLITH L-19

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - TYPICAL CHAMBER MONOLITH L - 25

COMPUTED BY: DATE:

CHECKED BY: DATE:

STATION 1+01.5 A TO 0+61.3 A

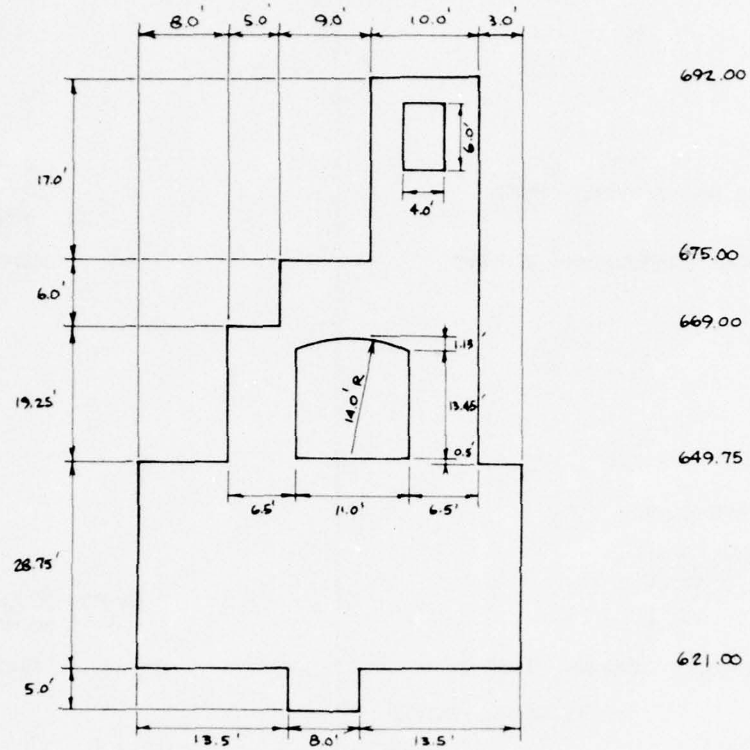


FIGURE A-4 LAND WALL - TYPICAL CHAMBER MONOLITH L-25

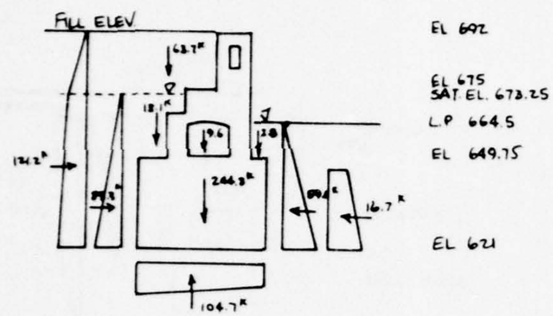
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL TYPICAL CHAMBER MONOLITH L-25

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{conc.}$	$[.1513][(649.75-621)(35)+(5)(8)+(669-649.75)(24)+(675-669)(19)+(692-675)(10)-(4)(6)-(14)(11)]$	244.3		19.19	4687
$W_{water,\ lock}$	$[.0625](664.5-649.75)(3)$	2.8		33.50	94
$W_{water,\ land}$	$[.0625][(673.25-669)(6)+(673.25-649.75)(8)]$	13.1		4.66	61
$W_{water,\ culv.}$	$[.0625](11)(14)$	9.6		20.00	192
W_{earth}	$[.1207][(692-673.25)(8)+(692-673.25)(5)+(692-675)(9)]+[.0754][(673.25-649.75)(8)+(673.25-669)(5)]$	63.7		9.24	589
$P_{water,\ lock}$	$[.0625](1/2)(664.5-621)^2$		-59.1	14.50	-857
$P_{water,\ land}$	$[.0625](1/2)(673.25-621)^2$		85.3	17.42	1486
P_{earth}	$[.1207][(692-673.25)^2(1/2)(.5)+(692-673.25)(.5)(673.25-621)]+[.0754](673.25-621)^2(1/2)(.5)$		121.2	26.98	3270
$P_{earth\ \&\ conc}$	$[.1513](649.75-647.75)(.5)(647.75-621)+[.0754](647.75-621)^2(1/2)(5)$		-17.5	9.94	-174
Uplift	$[.0625][(664.5-621)(35)+(1/2)(673.25-664.5)(35)]$	-104.7		16.97	-1777
		228.8	129.9		7571

$$e = \frac{7571}{228.8} = 33.09$$

PERCENT ACTIVE BASE $\dfrac{(35-33.09)(3)(100)}{35} = 16.37\%$

$$F.S. \frac{\Sigma\ \text{STABILIZING}\ M}{\Sigma\ \text{OVERTURNING}\ M} = \frac{7079.9}{6643.7} = 1.07$$

CASE I NORMAL OPERATIONS

SATURATION ELEV = 673.25
LOWER POOL IN LOCK

FIGURE A-4 LAND WALL TYPICAL CHAMBER MONOLITH L-25

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - TYPICAL CHAMBER MONOLITH L-25

COMPUTED BY: DATE:

CHECKED BY: DATE:

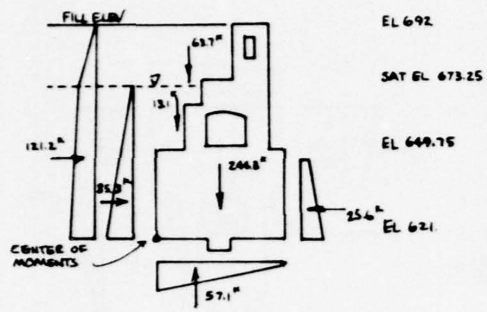
NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		244.3		19.19	4687
$W_{WATER, LAND}$		13.1		4.66	61
W_{EARTH}		63.7		9.24	589
$P_{WATER, LAND}$			85.3	17.42	1486
P_{EARTH}			121.2	26.98	3270
$P_{EARTH \,\&\, CONC.}$	$[.1513](2)(1/6)(26.75) + [.1207](26.75)^2(1/2)(.5)$		-25.6	9.62	-246
UPLIFT	$[.0625](673.25-621)(1/2)(35)$	-57.1		11.67	-666
		264.0	180.9		9181

$$e = \frac{9181}{264} = 34.77'$$

PERCENT ACTIVE BASE $\dfrac{(35-34.77)(3)(100)}{35} = 1.97\%$

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{6146.7}{6088.1} = 1.01$$



CASE II MAINTENANCE CONDITION

SATURATION ELEV = 673.25
LOCK DEWATERED

FIGURE A-4 LAND WALL - TYPICAL CHAMBER MONOLITH L-25

SUBJECT: LAND WALL TYPICAL CHAMBER MONOLITH L-25

COMPUTED BY: DATE:

CHECKED BY: DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \Sigma F_V \tan\phi + \text{Key Resistance}$$
$$= (228.8)(.5658) + (8)(.075)(144)$$
$$= 129.5 + 86.4$$
$$= 215.9$$

$$SSF = 215.9/129.9$$
$$= 1.66$$

CASE II MAINTENANCE CONDITION

$$R = (264)(.5658) + 86.4$$
$$= 149.4 + 86.4$$
$$= 235.8$$

$$SSF = 235.8/180.9$$
$$= 1.30$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \frac{2}{3}\frac{P}{e}$$
$$= \frac{(2)(228.8)}{(3)(35-33.09)}$$
$$= 79.8 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{2}{3}\frac{P}{e}$$
$$= \frac{(2)(264)}{(3)(35-34.77)}$$
$$= 765.2 \text{ KSF}$$

FIGURE A-4 LAND WALL - TYPICAL CHAMBER MONOLITH L-25

SUBJECT: LAND WALL - LOWER GATE MONOLITH L-33

COMPUTED BY:

CHECKED BY:

DATE

DATE

STATION 2+82.08 TO 2+35.55 B

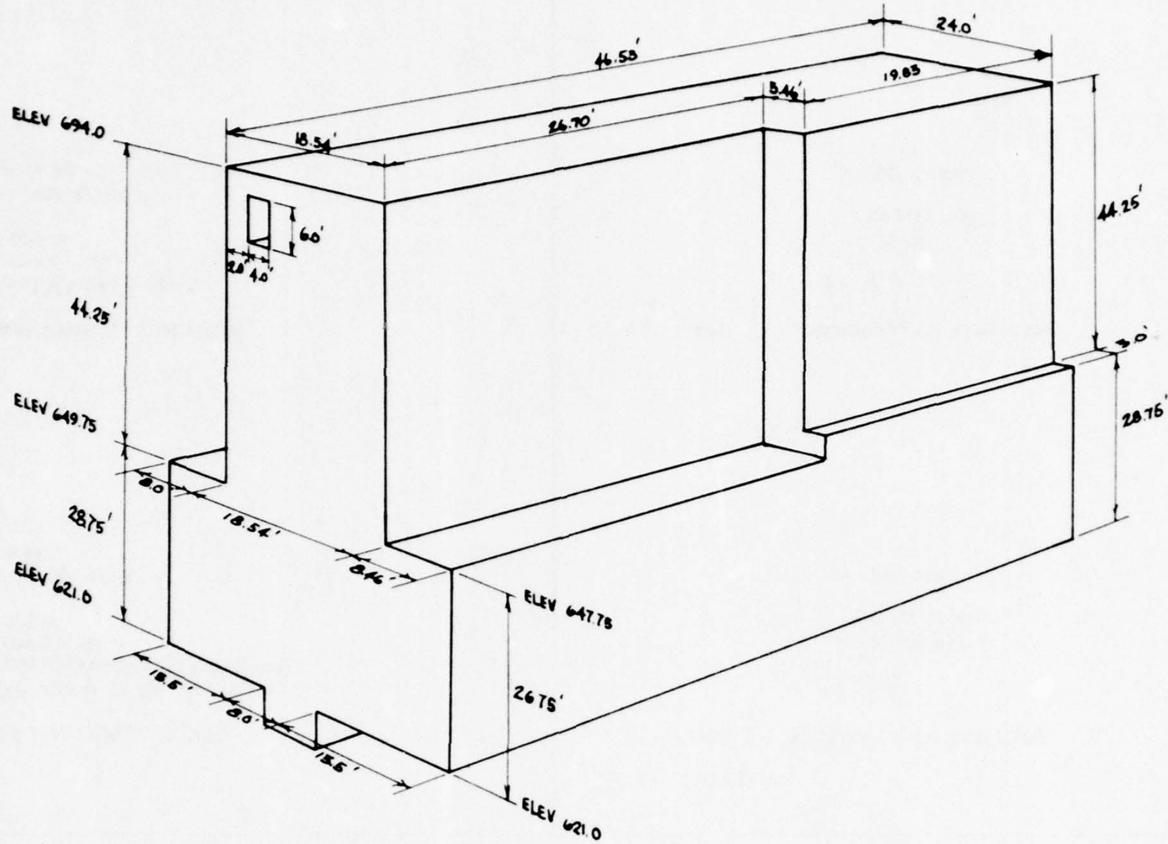


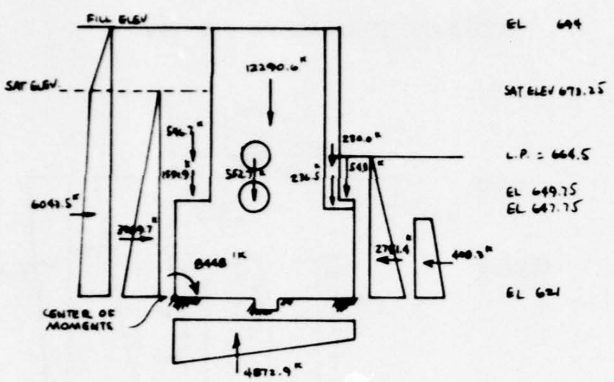
FIGURE A-5 LANDWALL - LOWER GATE MONOLITH L-33

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - LOWER GATE MONOLITH L-33

COMPUTED BY: DATE:

CHECKED BY: DATE:



CASE I NORMAL OPERATIONS

LOWER POOL BELOW AND IN LOCK
SAT. ELEV. = 673.25

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$	$[.1513][(694-621)(18.54)(46.53)+(28.75)(8)(46.53)+(26.75)(8.46)(46.53)+(5.46)(46.25)(19.83)+(5)(8)(46.53)+(3)(2)(19.83)-(2)(\pi)(5.5)^2(46.53)-(4)(6)(46.53)]$	12290.6		18.22	223985
$W_{WATER,\ LOCK}$	$[.0625](664.5-647.75)(8.46)(26.7)$	236.5		30.77	7277
	$[.0625](664.5-649.75)(3)(19.86)$	54.8		33.50	1836
$W_{WATER,\ LAND}$	$[.0625](673.25-649.75)(8)(46.53)$	546.7		4.00	2187
$W_{EARTH,\ LAND}$	$[.1207](694-673.25)(8)(46.53)+[.0754](673.25-649.75)(8)(46.53)$	1591.9		4.00	6368
$W_{WATER,\ CULVERT}$	$[.0625](2)(\pi)(5.5)^2(46.53)$	552.7		17.25	9534
$P_{WATER,\ LOCK}$	$[.0625](1/2)(664.5-621)^2(46.53)$		-2751.4	14.50	-39895
$P_{WATER,\ LAND}$	$[.0625](1/2)(673.25-621)^2(46.53)$		3969.7	17.42	69152
P_{EARTH}	$[.1207][(694-673.25)^2(1/2)(46.53)(.5)+(694-673.25)(.5)(673.25-621)(46.53)]+[.0754](1/2)(673.25-621)^2(.5)(46.53)$		6043.5	27.74	167647
$P_{EARTH\ \&\ CONC}$	$[.1513](647.75-645.75)(26.7)(645.75-621)+[.0754](645.75-621)^2(.5)(1/2)(26.7)$		-408.3	9.26	-3781
Uplift	$[.0625][(664.5-621)(35)(46.53)+(1/2)(673.25-664.5)(35)(46.53)]$	-48729		16.97	-82693
GATE V		2800		29.33	8212
GATE M					8448
		10680.3	6853.5		378277

$$e = \frac{378277}{10680.3} = 35.42$$

$$F.S. = \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{328689.5}{324657.4} = 1.01$$

FIGURE A-5 LAND WALL - LOWER GATE MONOLITH L-33

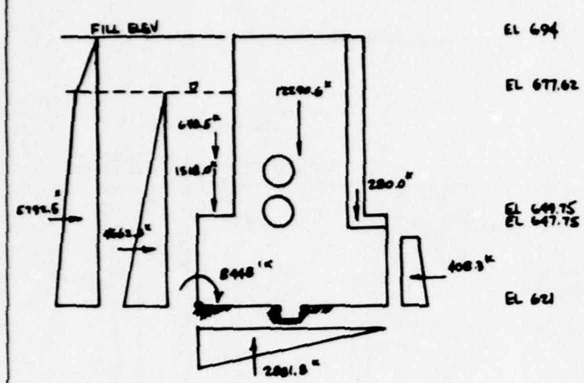
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - LOWER GATE MONOLITH L-33

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		12290.6		18.22	223985
$W_{WATER, LAND}$	$[.0625](677.62-649.75)(8)(46.53)$	648.5		4.00	2594
$W_{EARTH, LAND}$	$[.1207](694-677.62)(8)(46.53) + [.0754](677.62-649.75)(8)(46.53)$	1518.0		4.00	6072
$P_{WATER, LAND}$	$[.0625](1/2)(677.62-621)^2(46.53)$		4662.3	18.87	88001
$P_{EARTH, LAND}$	$[.1207][(1/2)(694-677.62)^2(.5)(46.53)+(694-677.62)(.5)(677.62-621)(46.53)]+[.0754](1/2)(677.62-621)^2(.5)(46.53)$		5792.5	27.74	160685
$P_{EARTH/CONC}$			-408.3	9.26	-3781
UPLIFT	$[.0625](677.62-621)(35)(46.53)$	-2881.8		11.67	-33630
GATE V		280.0		29.33	8212
GATE M					8448
		11855.3	10046.5		460586

$$e = \frac{460686}{11855.3} = 38.85$$

Percent active base: Resultant falls outside base

$$F.S. = \frac{\Sigma \text{ STABILIZING M}}{\Sigma \text{ OVERTURNING M}} = \frac{278766}{315918} = 0.88$$

CASE II MAINTENANCE CONDITION

LOCK DEWATERED
SATURATION ELEVATION 677.62

FIGURE A-5 LAND WALL - LOWER GATE MONOLITH L-33

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - LOWER GATE MONOLITH L-33

COMPUTED BY: | DATE:
CHECKED BY: | DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \Sigma F_V \tan\phi + \text{Key Resistance}$$
$$= (10680.3)(.5658) + (8)(46.53)(144)(.075)$$
$$= 6042.9 + 4020.2$$
$$= 10063.1$$

$$SSF = 10063.1 / 6853.5 =$$
$$= 1.47$$

CASE II MAINTENANCE CONDITION

$$R = (11855.3)(.5658) + 4020.2$$
$$= 6707.7 + 4020.2$$
$$= 10727.9$$

$$SSF = 10727.9 / 10046.5$$
$$= 1.07$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

Base Pressures very large

CASE II MAINTENANCE CONDITION

Base Pressure very large

FIGURE A-5 LAND WALL - LOWER GATE MONOLITH L-33

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - LOWER GUIDE WALL MONOLITH L-42

COMPUTED BY: DATE:

CHECKED BY: DATE:

STATION 6+82.00 B TO 6+34.00 B

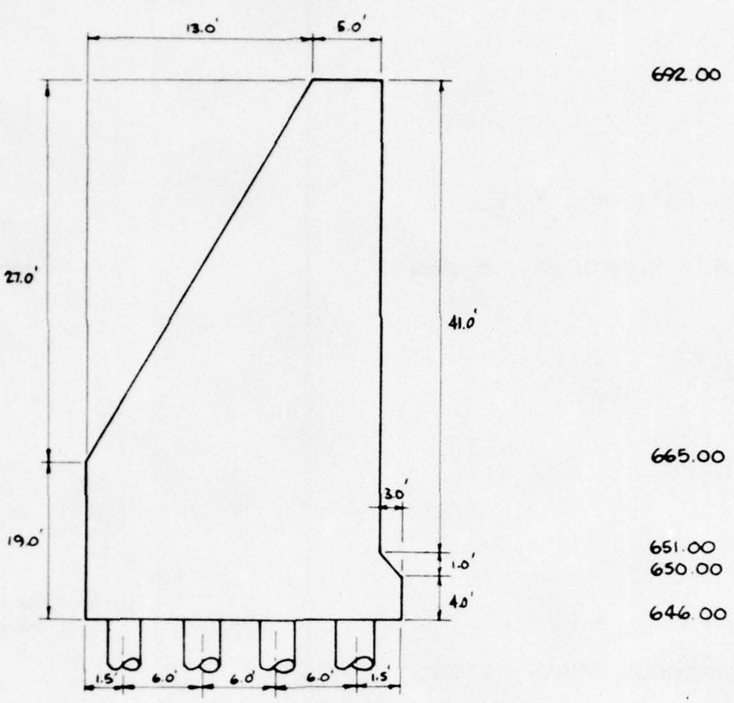


FIGURE A-6 LAND WALL - TYPICAL LOWER GUIDE WALL MONOLITH L-42

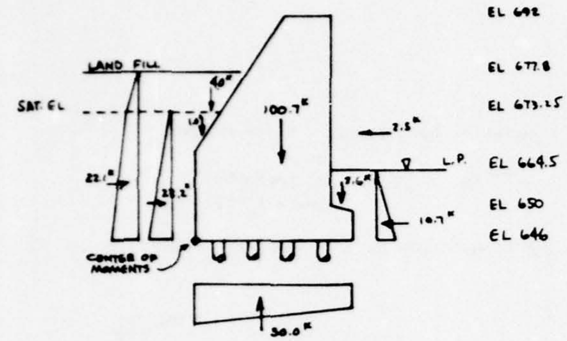
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LAND WALL - LOWER GUIDE WALL MONOLITH L-42

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513][(5)(46) + (1/2)(13)(27) + (19)(13) + (4)(3) + (1/2)(1)(3)]	100.7		10.44	1051
$W_{WATER, RIVER}$	[.0625][(664.5 - 651)(3) + (651 - 650)(1/2)(3)]	2.6		19.51	51
$W_{WATER, LAND}$	[.0625](1/2)(673.25 - 665)(3.97)	1.0		1.32	1
$W_{EARTH, LAND}$	[.1207][(677.8 - 673.25)(3.97) + (1/2)(677.8 - 673.25)(2.19)] + [.0754](1/2)(673.25 - 665)(3.97)]	4.0		2.52	10
$P_{WATER, RIVER}$	$[.0625](1/2)(664.5-646)^2$		-10.7	6.17	-66
$P_{WATER, LAND}$	$[.0625](1/2)(673.25-646)^2$		23.2	9.08	211
P_{EARTH}	$[.1207][(677.8-673.25)^2(1/2)(.5) + (677.8-673.25)(.5)(673.25-646)] + [.0754](673.25-646)^2(1/2)(.5)$		22.1	12.08	267
UPLIFT	[.0625][(664.5 - 646)(21) + (1/2)(673.25 - 664.5)(21)]	-30.0		9.83	295
IMPACT			-2.5	23.50	-59
		78.3	32.1		1761

$$e = \frac{1761}{78.3} = 22.49$$

CASE I NORMAL OPERATIONS

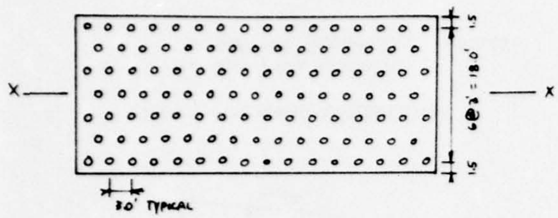
LOWER POOL IN RIVER
SATURATION ELEVATION 673.25

FIGURE A-6 LAND WALL - TYPICAL LOWER GUIDE WALL MONOLITH L-42

WES FORM NO. 1253A DECEMBER 1964

SUBJECT	COMPUTED BY	DATE
LAND WALL - LOWER GUIDE WALL MONOLITH L-42	CHECKED BY	DATE

HORIZONTAL PILE LOADS



ALLOWABLE LOAD PER PILE IN HORIZONTAL = 8^K

$F_H = 32.1^K$ per foot

$(32.1)(48) = 1540.8^K = F_{H\,TOTAL}$

load per pile = $1540.8 / 109 = 14.14^K > 8^K$

BASE PILE PRESSURE

I of the pile group about xx axis is $3115.5\ ft^4$

$$f = \frac{P}{A} + \frac{Mc}{I}$$

$$= \frac{(78.3)(48)}{(.7854)(109)} + \frac{(78.3)(48)(22.49-10.5)(9)}{3115.5}$$

$$= 43.9 + 130.2$$

$$= 174.1\ KSF$$

ALLOWABLE PER PILE = 100 K/pile

$174.1 \frac{K}{SF} \times .7854 \frac{SF}{pile} = 136.7\ K/pile > 100\ K/pile$

FIGURE A-6 LAND WALL - TYPICAL LOWER GUIDE WALL MONOLITH L-42

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - UPPER GATE MONOLITH (600' LOCK) M-5

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

STATION 4+23.73 A TO 3+75.73 A

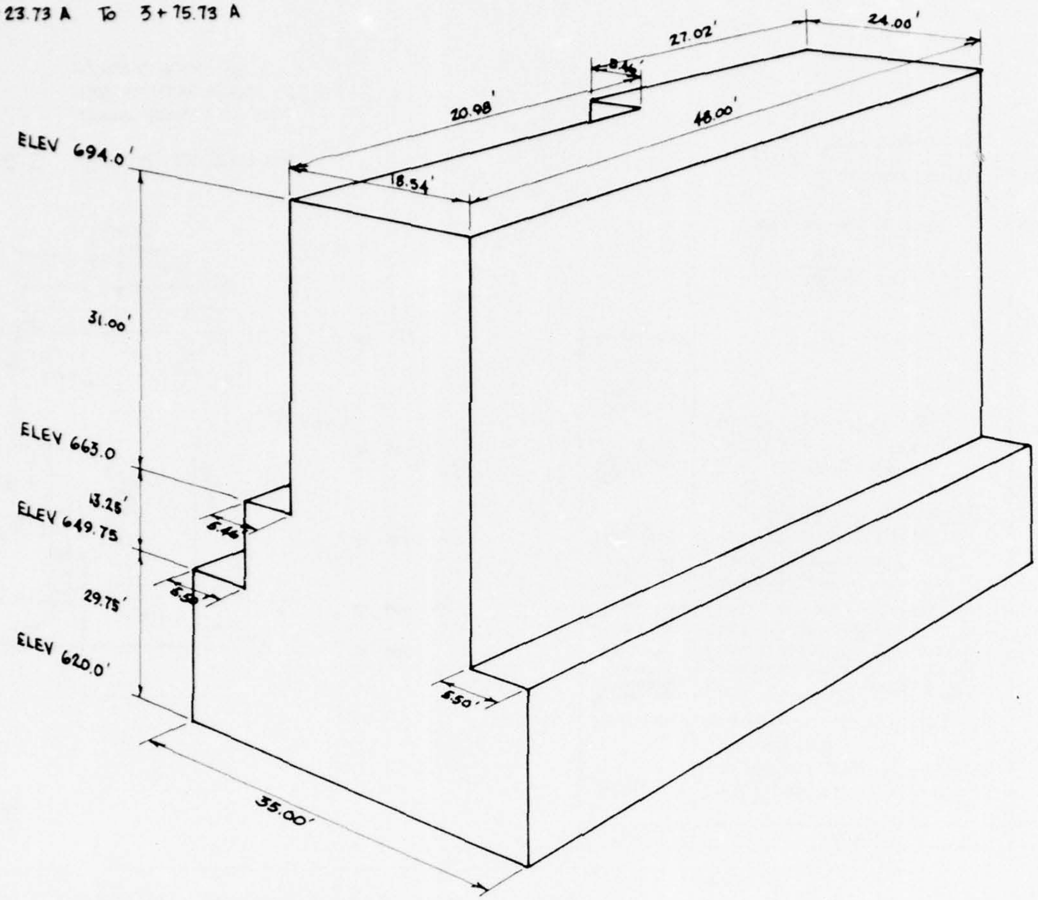
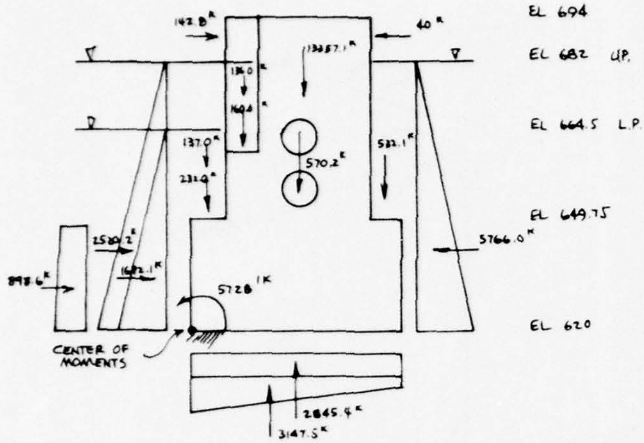


FIGURE A-7 MIDDLE WALL - UPPERG GATE MONOLITH (600' LOCK) M-5

SUBJECT: MIDDLE WALL UPPER GATE MONOLITH (600' LOCK) M-5

COMPUTED BY: | DATE:
CHECKED BY: | DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[.1513][(24)(48)(44.25) - (5.46)(20.98)(31) + (35)(48)(29.75) - (2)(\pi)(5.5)^2(48)]$	13357.1		17.59	234951
$W_{WATER, RIVER}$	$[.0625](682-649.75)(5.5)(48)$	532.1		32.25	17160
$W_{WATER, LOCK}$ (ABOVE GATE)	$[.0625](682-663)(5.46)(20.98)$	136.0		8.23	1119
(ABOVE GATE)	$[.0625](682-649.75)(5.5)(20.98)$	232.6		2.75	640
(IN LOCK)	$[.0625](664.5-649.75)(5.5)(27.02)$	137.0		2.75	377
$W_{WATER, CULVERT}$	$[.0625](2)(\pi)(5.5)^2(48)$	570.2		20.25	11547
$P_{WATER, RIVER}$	$[.0625](1/2)(682-620)^2(48)$		-5766.0	20.67	-119183
$P_{WATER, LOCK}$ (ABOVE GATE)	$[.0625](1/2)(682-620)^2(20.98)$		2520.2	20.67	52093
(IN LOCK)	$[.0625](1/2)(664.5-620)^2(27.02)$		1672.1	14.83	24797
$P_{EARTH \& CONC}$ IN LOCK	$[[.1513](649.75-647.75)(5)(647.75-620) + [.0754](647.75-620)^2(1/2)(.5)](48)$		898.6	10.28	9238
U_{PLIFT} (ABOVE GATE)	$[.0625](682-620)(35)(20.98)$	-2845.4		17.50	-49795
(IN LOCK)	$[.0625][(664.5-620)(35)(27.02) + (1/2)(682-664.5)(35)(27.02)]$	-3147.5		18.46	-58103
GATE S			142.8	72.67	10377
GATE V		160.0		8.30	1328
GATE M					-5728
IMPACT, RIVER			-40.0	67.00	-2680
		9132.1	-572.3		128138

$$e = \frac{128138}{9132.1} = 14.03$$

$$\text{PERCENT ACTIVE BASE} = \frac{(14.03)(3)(100)}{35} = 100\%$$

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{363627}{235489} = 1.54$$

CASE I NORMAL OPERATIONS

UPPER POOL IN RIVER
UPPER POOL ABOVE LOCK
LOWER POOL IN LOCK

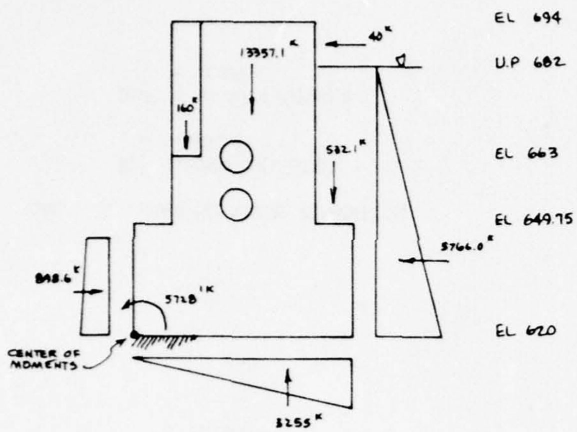
FIGURE A-7 MIDDLE WALL - UPPER GATE MONOLITH (600' LOCK) M-5

SUBJECT

MIDDLE WALL - UPPER GATE MONOLITH (600' LOCK) M-5

COMPUTED BY: DATE:
CHECKED BY: DATE:

NOT TO SCALE



CASE II MAINTENANCE CONDITION

LOCK DEWATERED
UPPER POOL IN RIVER

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$		13357.1		17.59	234951
$W_{WATER, RIVER}$		532.1		32.25	17160
$P_{WATER, RIVER}$			-5766.0	20.67	-119183
$P_{EARTH \,\&\, CONC.}$			898.6	10.28	9238
UPLIFT	$[.0625](682-620)(1/2)(35)(48)$	-3255.0		23.33	-75939
GATE V		160.0		8.30	1328
GATE M					-5728
$I_{MPACT, RIVER}$			-40.0	67.00	-2680
		10794.2	-4907.4		59147

$$e = \frac{59147}{10794.2} = 5.48'$$

PERCENT ACTIVE BASE $\frac{(5.48)(3)(100)}{35} = 46.97\%$

$$F.S. = \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{262677}{203530} = 1.29$$

FIGURE A-7 MIDDLE WALL - UPPER GATE MONOLITH (600' LOCK) M-5

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - UPPER GATE MONOLITH (600' LOCK) M-5

COMPUTED BY: | DATE:
CHECKED BY: | DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \sum F_v \tan\phi + \text{Key Resistance}$$
$$= (9132.1)(.5658) + 0$$
$$= 5166.9$$

$$SSF = 5166.9 / 572.3$$
$$= 9.03$$

CASE II MAINTENANCE CONDITION

$$R = (10794.2)(.5658) + 0$$
$$= 6107.4$$

$$SSF = 6107.4 / 4907.4$$
$$= 1.24$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{9132.1}{(35)(48)} + \frac{(9132.1)(17.5-14.03)(17.5)(12)}{(48)(35)^3}$$
$$= 5.44 + 3.23$$
$$= 8.67 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = \left(\frac{2}{3}\frac{P}{e}\right) / L$$
$$= \frac{(2)(10794.2)}{(3)(5.48)} / 48$$
$$= 27.36 \text{ KSF}$$

FIGURE A-7 MIDDLE WALL - UPPER GATE MONOLITH (600' LOCK) M-5

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - TYPICAL UPPER CHAMBER MONOLITH M-7

COMPUTED BY | DATE

CHECKED BY | DATE

STATION 3+57.67 A TO 3+15.73 A

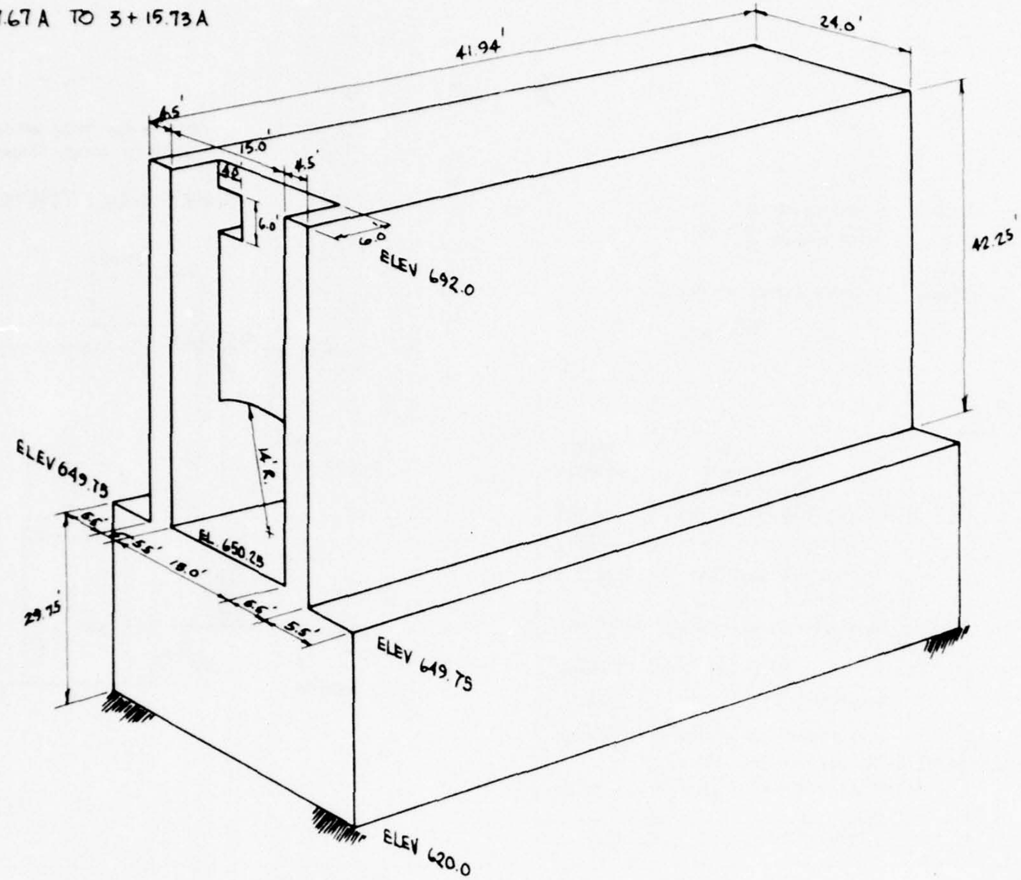


FIGURE A-8 MIDDLE WALL - TYPICAL UPPER CHAMBER MONOLITH M-7

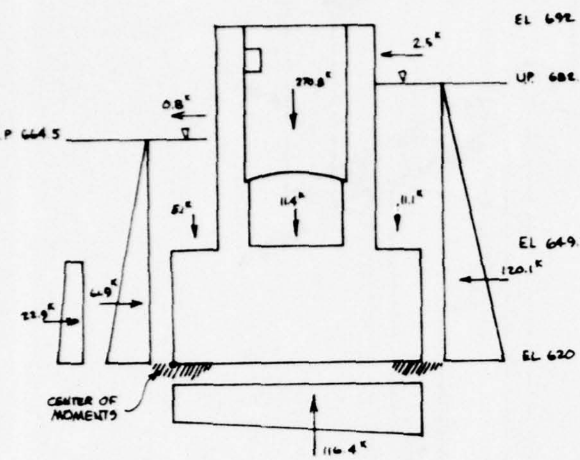
WES FORM NO. 1253A DECEMBER 1964

SUBJECT

MIDDLE WALL - TYPICAL UPPER CHAMBER MONOLITH M-7

COMPUTED BY | DATE
CHECKED BY | DATE

NOT TO SCALE



CASE I NORMAL OPERATIONS

LOWER POOL IN LOCK
UPPER POOL IN RIVER

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513][(24)(42.25) + (35)(29.75) − (6)(4) − (13)(14) − (27.75)(15)(6)/41.94]	270.8		17.41	4715
$W_{WATER\ IN\ RIVER}$	[.0625](682 − 649.75)(5.5)	11.1		32.25	358
$W_{WATER\ IN\ LOCK}$	[.0625](664.5 − 649.75)(5.5)	5.1		2.75	14
$W_{WATER\ IN\ CULVERT}$	[.0625](14)(13)	11.4		17.50	200
$P_{CONC\ \&\ EARTH}$	[.1513](649.75 − 647.75)(.5)(647.75 − 620)(1) + [.0754](647.75 − 620)²(.5)(1/2)		22.9	10.95	251
$P_{WATER\ IN\ RIVER}$	[.0625](682 − 620)²(1/2)		−120.1	20.67	−2482
$P_{WATER\ IN\ LOCK}$	[.0625](664.5 − 620)²(1/2)		61.9	14.83	918
UPLIFT	[.0625][(664.5 − 620)(35) + (1/2)(682 − 664.5)(35)]	−116.4		18.46	−2149
HAWSER	0.8 KIPS/FT.		−0.8	49.50	−40
IMPACT	2.5 KIPS/FT.		−2.5	67.00	−168
		182.0	−38.6		1617

$e = \frac{1617}{182} = 8.88'$

PERCENT ACTIVE BASE $= \frac{8.88(3)(100)}{35} = 76.11\%$

$F.S. = \frac{\sum \text{STABILIZING M.}}{\sum \text{OVERTURNING M.}} = \frac{6456}{4839} = 1.33$

FIGURE A-8 MIDDLE WALL - TYPICAL UPPER CHAMBER MONOLITH M-7

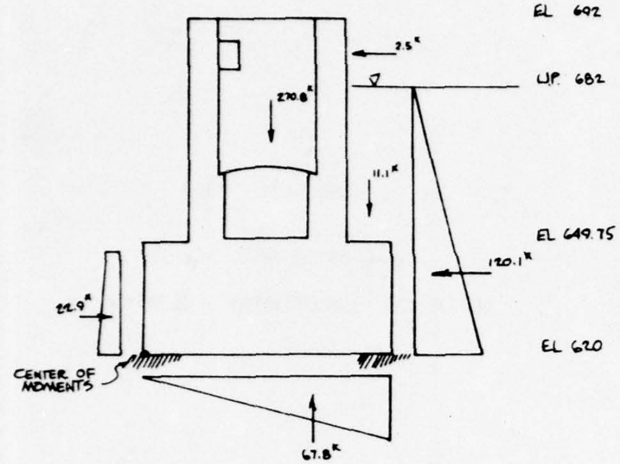
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - TYPICAL UPPER CHAMBER MONOLITH M-7

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		270.8		17.40	4712
$W_{WATER\ IN\ RIVER}$		11.1		32.25	358
$P_{EARTH\ \&\ CONC}$			22.9	10.95	251
$P_{WATER\ IN\ RIVER}$			-120.1	20.67	-2482
UPLIFT	[.0625](1/2)(682-620)(35)	-67.8		23.33	-1582
IMPACT			-2.5	67.00	-168
		214.1	-99.7		1089

$$e = \frac{1089}{214.1} = 5.09'$$

$$\text{PERCENT ACTIVE BASE} = \frac{5.09(3)(100)}{35} = 43.63\%$$

$$F.S. = \frac{\sum \text{STAB. M.}}{\sum \text{OVRT. M.}} = \frac{5321}{4232} = 1.26$$

CASE II MAINTENANCE CONDITION

UPPER POOL IN RIVER
LOCK DEWATERED

FIGURE A-8 MIDDLE WALL - TYPICAL UPPER CHAMBER MONOLITH M-7

WES FORM NO. 1253A DECEMBER 1964

SUBJECT MIDDLE WALL - TYPICAL UPPER CHAMBER MONOLITH M-7

COMPUTED BY | DATE

CHECKED BY | DATE

SLIDING

CASE I NORMAL OPERATIONS

$$R = \Sigma F_v \tan\phi + \text{KEY RESISTANCE}$$
$$= (182.0)(.5658) + 0$$
$$= 103.0$$

$$SSF = 103.0/38.6 = 2.67$$

CASE II MAINTENANCE CONDITION

$$R = (214.1)(.5658) + 0$$
$$= 112.1$$

$$SSF = 112.1/99.7 = 1.12$$

BASE PRESSURES

CASE I NORMAL OPERATIONS

$$f = 2/3 \frac{P}{e}$$
$$= \frac{(2)(182.0)}{(3)(8.88)}$$
$$= 13.67 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = 2/3 \frac{P}{e}$$
$$= \frac{(2)(214.1)}{(3)(5.09)}$$
$$= 28.04 \text{ KSF}$$

FIGURE A-8 MIDDLE WALL - TYPICAL UPPER CHAMBER MONOLITH M-7

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - UPPER GATE MONOLITH (360' LOCK) M-10

COMPUTED BY:

DATE

CHECKED BY:

DATE

STATION 2+53.0 A TO 2+11.95 A

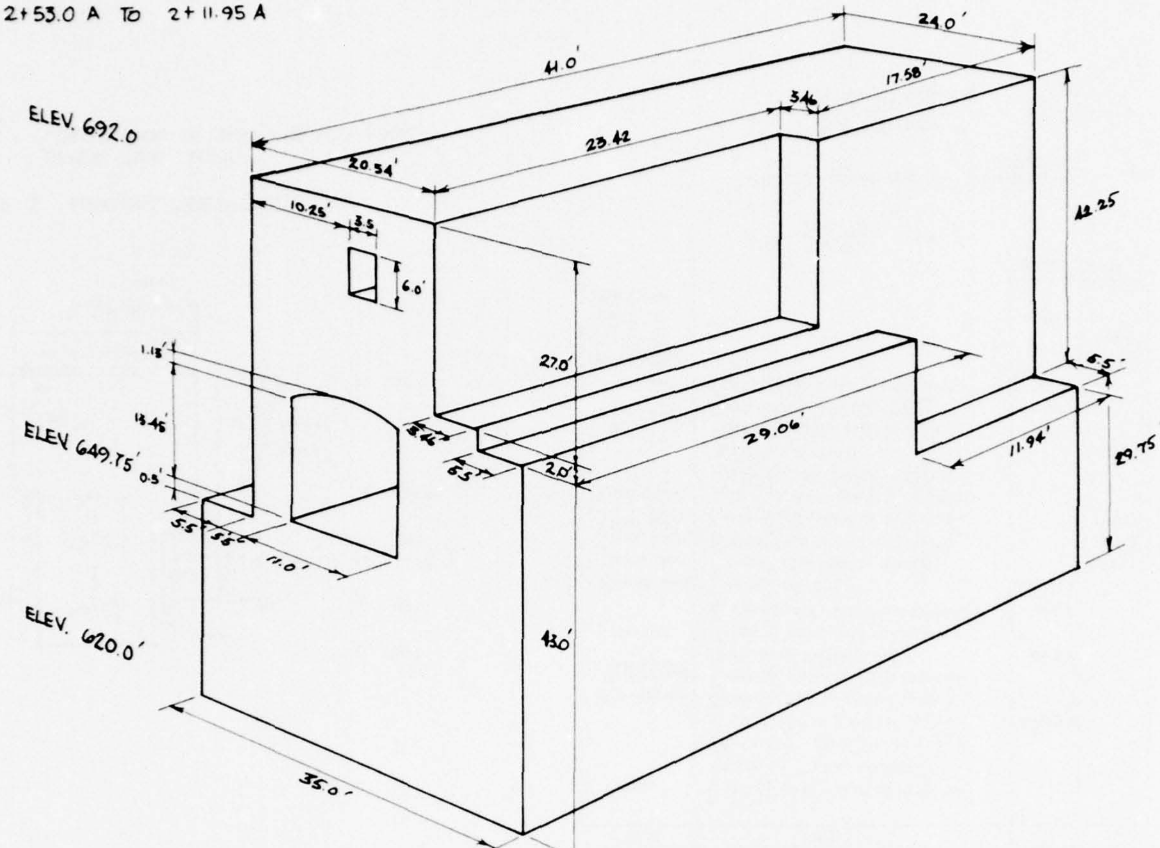


FIGURE A-9 MIDDLE WALL - UPPER GATE MONOLITH (360' LOCK) M-10

WES FORM NO. 1253A DECEMBER 1964

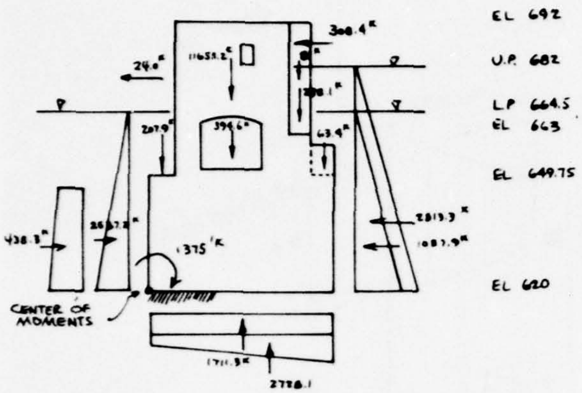
SUBJECT

MIDDLE WALL - UPPER GATE MONOLITH (360' LOCK) M-10

COMPUTED BY: | DATE:

CHECKED BY: | DATE:

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENTS
$W_{CONC.}$	[.1513][(24)(41)(42.25) + (35)(41)(29.75) + (663 - 649.75)(55)(41 - 11.91) - (3.46)(23.42)(27) - (11)(14)(41) - (3.5)(6)(41)]	11653.2		17.76	206961
$W_{WATER,\ LOCK}$	[.0625](664.5 - 649.75)(5.5)(41)	207.9		2.75	572
$W_{WATER,\ RIVER}$ (ABOVE GATE)	[.0625][(682 - 665)(23.42)(3.46) + (682 - 663)(23.42)(5.5)]	238.1		30.65	7298
(IN LOCK)	[.0625][(664.5 - 663)(17.58 - 11.94)(5.5) + (664.5 - 649.75)(11.94)(5.5)]	63.4		32.25	2045
$W_{WATER,\ CULVERT}$	[.0625](11)(14)(41)	394.6		16.50	6511
$P_{WATER,\ LOCK}$	[.0625](½)(664.5 - 620)²(41)		2537.2	14.83	37627
$P_{WATER,\ RIVER}$ (ABOVE GATE)	[.0625](½)(682 - 620)²(23.42)		-2813.3	20.67	-58151
(IN LOCK)	[.0625](½)(664.5 - 620)²(17.58)		-1087.9	14.83	-16134
$P_{EARTH\ \&\ CONC}$	[.1513](649.75 - 647.75)(.5)(647.75 - 620)(23.42) + [.0754](½)(647.75 - 620)²(.5)(23.42)		438.3	10.63	4659
U_{PLIFT} (ABOVE GATE)	[.0625][(664.5 - 620)(35)(23.42) + (682 - 664.5)(½)(35)(23.42)]	-2728.1		18.46	-50361
(IN LOCK)	[.0625](664.5 - 620)(35)(17.58)	-1711.3		17.50	-29948
HAWSER, INNER LOCK			-24.0	49.50	-1188
GATE S			-308.4	50.67	-15627
GATE V		81.0		26.52	2148
GATE M					1375
		8198.8	-1258.1		97787

$$e = \frac{97787}{8198.8} = 11.93'$$

$$\text{PERCENT ACTIVE BASE} = \frac{(11.93)(3)(100)}{35} = 100\%$$

$$FS = \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{269196}{171409} = 1.57$$

CASE I NORMAL OPERATION

UPPER POOL ABOVE
LOWER POOL IN 360' AND 600' LOCKS

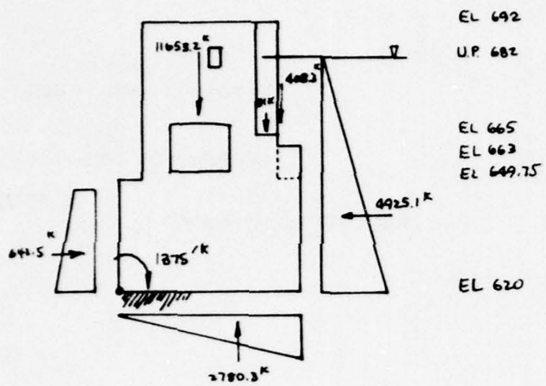
FIGURE A-9 MIDDLE WALL - UPPER GATE MONOLITH (360' LOCK) M-10

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - UPPER GATE MONOLITH (360' LOCK) M-10

COMPUTED BY: | DATE:
CHECKED BY: | DATE:

NOT TO SCALE



CASE II MAINTENANCE CONDITION

UPPER POOL ABOVE AND IN 360' LOCK
600' LOCK DEWATERED

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{conc}		11653.2		17.76	206961
$W_{water, river}$	[.0625][(682-665)(23.42)(3.46) + (682-663)(29.06)(5.5) + (682-649.75)(11.94)(5.5)]	408.3		31.31	12784
$P_{water, river}$	[.0625](682-620)^2(1/2)(41)		-4925.1	20.67	-101802
$P_{earth\ \&\ conc}$	[.1513](2)(1/2)(27.75)(23.42)+[.1207][1/2](5)(27.75)^2(23.42)		642.5	9.95	6393
Uplift	[.0625](682-620)(1/2)(35)(41)	-2780.3		23.33	-64864
Gate K		81.0		26.52	2148
Gate M					1375
		9362.2	-4282.6		62995

$$e = \frac{62995}{9362.2} = 6.72$$

$$\text{PERCENT ACTIVE BASE} = \frac{(6.72)(3)(100)}{35} = 57.60\%$$

$$F.S. = \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{229661}{166666} = 1.38$$

FIGURE A-9 MIDDLE WALL - UPPER GATE MONOLITH (360' LOCK) M-10

SUBJECT: MIDDLE WALL - UPPER GATE MONOLITH (360' LOCK) M-10

COMPUTED BY: | DATE:
CHECKED BY: | DATE:

SLIDING — NORMAL OPERATION

CASE I

$$R = \Sigma F_v \tan\phi + \text{Key Resistance}$$
$$= (8198.8)(.5658) + 0$$
$$= 4638.9$$

$$SSF = 4638.9/1258.1$$
$$= 3.68$$

MAINTENANCE CONDITION

CASE II

$$R = (9362.2)(.5658) + 0$$
$$= 5297.1$$

$$SSF = 5297.1/4282.6$$
$$= 1.24$$

BASE PRESSURE — NORMAL OPERATION

CASE I

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{8198.8}{(41)(35)} + \frac{(8198.8)(17.5-11.93)(17.5)(12)}{(41)(35)^3}$$
$$= 5.71 + 5.46$$
$$= 11.17 \text{ KSF}$$

MAINTENANCE CONDITION

CASE II

$$f = \frac{2P}{3e}/L$$
$$= \frac{(2)(9362.2)}{(3)(6.72)}/41$$
$$= 22.65 \text{ KSF}$$

FIGURE A-9 MIDDLE WALL UPPER GATE MONOLITH (360' LOCK) M-10

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - TYPICAL CHAMBER MONOLITH M-13 | COMPUTED BY: | DATE: | CHECKED BY: | DATE:

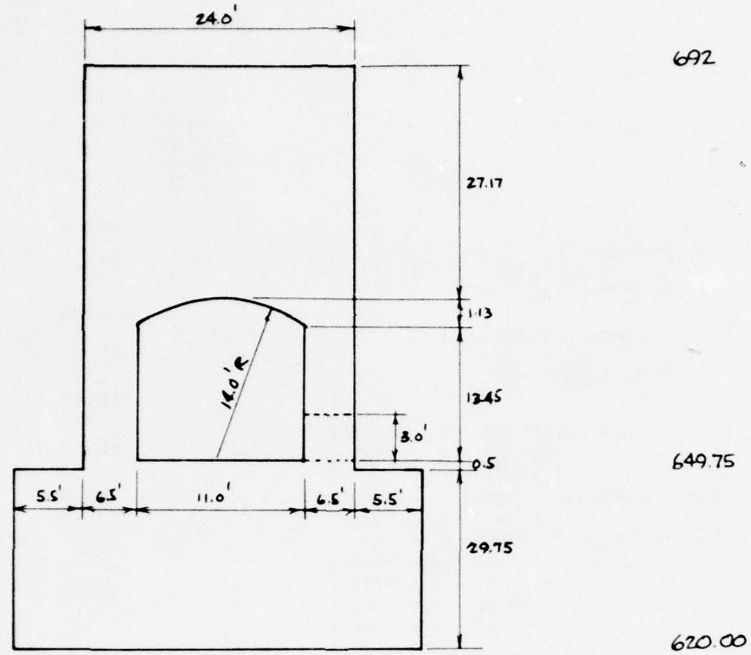


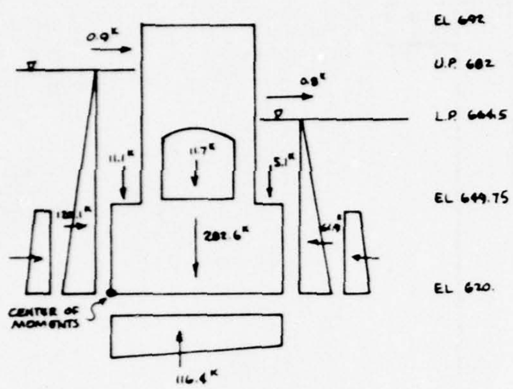
FIGURE A-10 MIDDLE WALL TYPICAL CHAMBER MONOLITH M-13

SUBJECT: MIDDLE WALL TYPICAL CHAMBER MONOLITH M-13

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE I NORMAL OPERATIONS

UPPER POOL IN INNER LOCK
LOWER POOL IN OUTER LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513][(24)(42.25) + (35)(29.75) − (13)(14) − (3)(3)(4)(6.5)/(43.63)]	282.6		17.53	4954
$W_{WATER,\ OUTER\ LOCK}$	[.0625](664.5 − 649.75)(5.5)	5.1		32.25	164
$W_{WATER,\ INNER\ LOCK}$	[.0625](682 − 649.75)(5.5)	11.1		2.75	31
$W_{WATER,\ CULVERT}$	[.0625][(13)(14) + (3)(3)(4)(6.5)/43.63]	11.7		17.24	202
$P_{WATER,\ OUTER\ LOCK}$	[.0625](664.5 − 620)²(1/2)		−61.9	14.83	−918
$P_{WATER,\ INNER\ LOCK}$	[.0625](682 − 620)²(1/2)		120.1	20.67	2482
$P_{EARTH\ \&\ CONC}$ (INNER & OUTER LOCK)	THE PRESSURES CANCEL EACH OTHER				
UPLIFT	[.0625][(664.5 − 620)(35) + (1/2)(682 − 664.5)(35)]	−116.4		16.54	−1926
HAWSER	0.8 K/FT		0.8	49.50	40
IMPACT	40 / 43.63 = 0.92		0.9	67.00	60
		194.1	59.9		5089

$$e = \frac{5089}{194.1} = 26.22$$

$$\text{PERCENT ACTIVE BASE} = \frac{(35 - 26.22)(3)(100)}{35} = 75.26\%$$

$$F.S. = \frac{\sum \text{STABILIZING M}}{\sum \text{OVERTURNING M}} = \frac{6627}{4923} = 1.35$$

FIGURE A-10 MIDDLE WALL – TYPICAL CHAMBER MONOLITH M-13

WES FORM NO. 1253A DECEMBER 1964

SUBJECT MIDDLE WALL - TYPICAL CHAMBER MONOLITH M-13

COMPUTED BY | DATE

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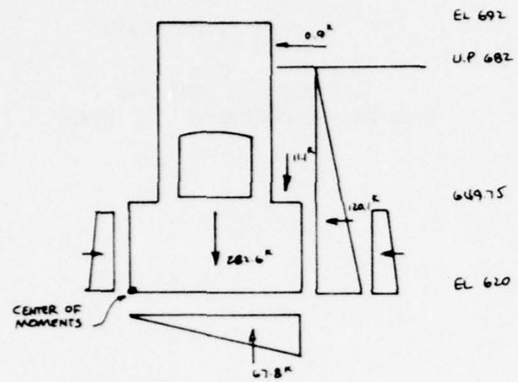
NOT TO SCALE



CASE II MAINTENANCE CONDITION

UPPER POOL IN OUTER LOCK
INNER LOCK DEWATERED

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		281.6		17.53	4954
$W_{WATER, OUTER LOCK}$	$[.0625](682-649.75)(5.5)$	11.1		32.25	358
$P_{WATER, OUTER LOCK}$	$[.0625](682-620)^2(1/2)$		-120.1	20.67	-2482
$P_{EARTH \& CONC}$ (INNER & OUTER)	THE PRESSURES CANCEL EACH OTHER				
UPLIFT	$[.0625](1/2)(682-620)(35)$	-67.8		23.33	-1582
IMPACT			-0.9	67.00	-60
		225.9	-121.0		1188

$$e = \frac{1188}{225.9} = 5.26'$$

$$\text{PERCENT ACTIVE BASE} = \frac{(5.26)(3)(100)}{35} = 45.08\ \%$$

$$F.S. \ \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{5304}{4124} = 1.29$$

FIGURE A-10 MIDDLE WALL - TYPICAL CHAMBER MONOLITH M-13

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - TYPICAL CHAMBER MONOLITH M-13

COMPUTED BY: DATE:
CHECKED BY: DATE:

SLIDING

CASE I NORMAL OPERATIONS

$R = \Sigma F_V \tan\phi$ + Key Resistance

$= (194.1)(.5658) + 0$

$= 109.8$

$SSF = 109.8/59.9$

$= 1.83$

CASE II MAINTENANCE CONDITION

$R = (225.9)(.5658) + 0$

$= 127.8$

$SSF = 127.8/121.0$

$= 1.06$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$f = \frac{2}{3}\frac{P}{e}$

$= \frac{(2)(194.1)}{(3)(8.78)}$

$= 14.73$ KSF

CASE II MAINTENANCE CONDITION

$f = \frac{2}{3}\frac{P}{e}$

$= \frac{(2)(225.9)}{(3)(5.26)}$

$= 28.63$ KSF

FIGURE A-10 MIDDLE WALL - TYPICAL CHAMBER MONOLITH M-13

SUBJECT: MIDDLE WALL - LOWER GATE MONOLITH (360' LOCK) M-20

COMPUTED BY: | DATE:
CHECKED BY: | DATE:

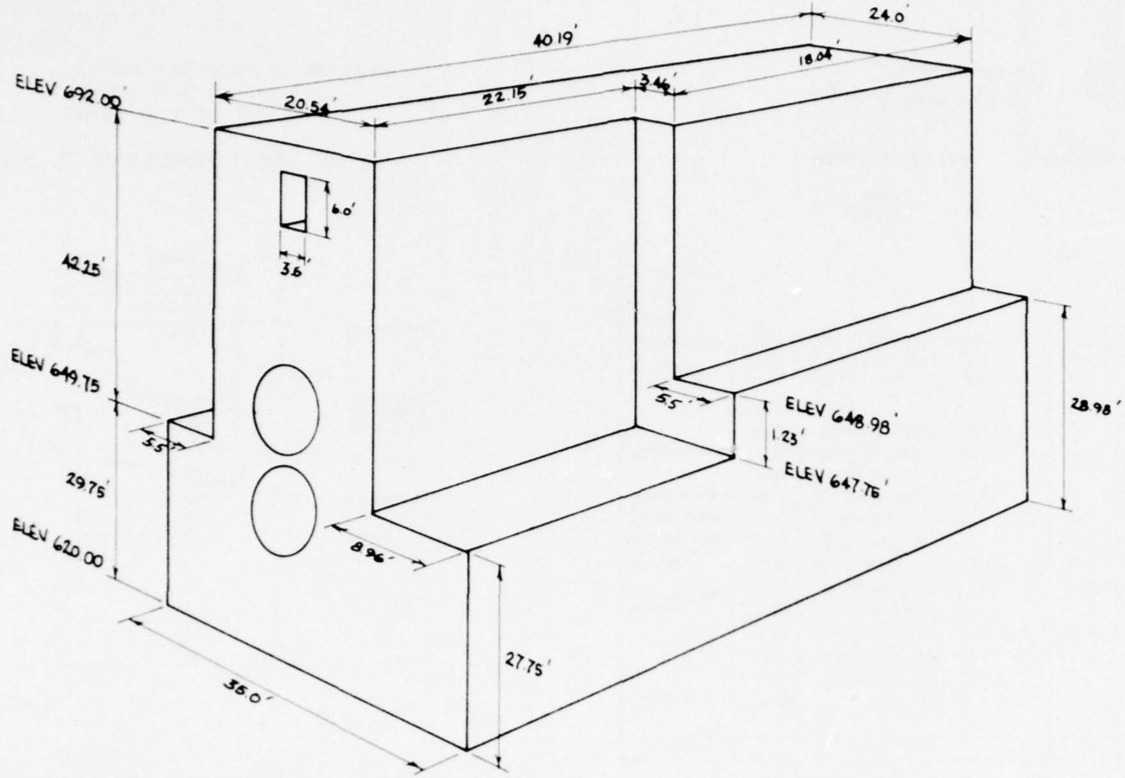


FIGURE A-11 MIDDLE WALL - LOWER GATE MONOLITH (360' LOCK) M-20

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - LOWER GATE MONOLITH (360' LOCK) M-20

COMPUTED BY: DATE:

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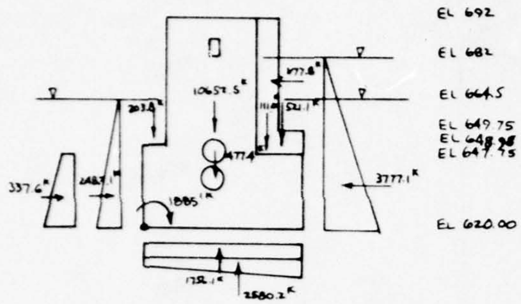
NOT TO SCALE



CASE I MAINTENANCE CONDITION

UPPER POOL IN 360' LOCK
LOWER POOL IN 600' LOCK
LOWER POOL BELOW 360' LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513][(29.75)(5.5)(40.19) + (20.54)(72)(40.19) + (44.25)(3.46)(18.04) + (8.96)(27.75)(40.19) + (5.5)(18.04)(1.23) (2)(π/4)(11)²(40.19) - (3.5)(6)(40.19)]	10652.5		17.05	181625
W_{WATER} (RIVER SIDE)	[.0625][(682 - 647.75)(8.96)(22.15) + (664.5 - 648.98)(5.5)(18.04)]	521.1		30.67	51982
W_{WATER} (LAND SIDE)	[.0625](664.5 - 649.75)(5.5)(40.19)	203.8		2.75	560
$W_{WATER, CULVERT}$	[.0625](2)(π/4)(11)²(40.19)	477.4		16.50	7877
$P_{WATER, LAND SIDE}$	[.0625](½)(664.5 - 620)²(40.19)		2487.1	14.83	36884
$P_{WATER, RIVER SIDE}$	[.0625][(½)(664.5 - 620)²(18.04) + (½)(682 - 620)²(22.15)]		-3777.1	18.94	-71538
$P_{EARTH \& CONC}$	[.1513](649.75 - 647.75)(.5)(647.75 - 620)(18.04) + [.0754](647.75 - 620)²(½)(.5)(18.04)		337.6	10.29	3474
UPLIFT (IN LOCKS)	[.0625][(664.5 - 620)(35)(22.15) + (½)(682 - 664.5)(35)(22.15)]	-2580.2		18.46	-47630
(BELOW GATE)	[.0625](664.5 - 620)(35)(18.04)	-1756.1		17.50	-30732
GATE V		111.0		27.77	3082
GATE S			-877.8	42.05	-36911
GATE M					1885
		7629.5	-1830.2		100558

$$e = \frac{100558}{7629.5} = 13.18$$

$$\text{PERCENT ACTIVE BASE}: \frac{(13.18)(3)(100)}{35} = 100\%$$

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{287369}{186811} = 1.54$$

FIGURE A-11 MIDDLE WALL - LOWER GATE MONOLITH (360' LOCK) M-20

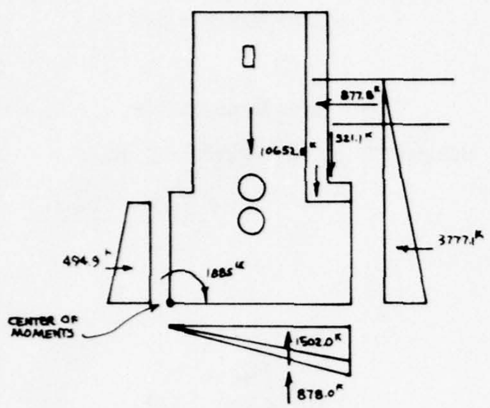
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: MIDDLE WALL - LOWER GATE MONOLITH (360' LOCK) M-20

COMPUTED BY | DATE

CHECKED BY | DATE

NOT TO SCALE



CASE II MAINTENANCE CONDITION

600' LOCK DEWATERED
UPPER POOL IN 360' LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		10652.5		17.05	181625
W_{WATER}, RIVER SIDE		521.1		30.67	51982
P_{WATER}, RIVER SIDE			-3777.1	18.94	-71538
P_{EARTH} & CONC.	$[.1513](2)(.5)(27.75)(18.04)+[.1207](27.75)^2(\frac{1}{2})(.5)(18.04)$		494.9	9.95	4924
UPLIFT					
(IN LOCKS)	$[.0625](682-620)(35)(\frac{1}{2})(22.15)$	-1502.0		23.33	-35042
(below LOCKS)	$[.0625](664.50-620)(35)(\frac{1}{2})(18.04)$	-878.0		23.33	-20484
GATE V		110.0		27.77	3082
GATE S			-877.8	42.05	-36911
GATE M					1885
		8903.6	-4160		79523

$$e = \frac{79523}{8903.6} = 8.93$$

$$\text{PERCENT ACTIVE BASE} = \frac{(8.93)(3)(100)}{35} = 76.54\%$$

$$\text{F.S.} \; \frac{\Sigma \text{ STABILIZING M}}{\Sigma \text{ OVERTURNING M}} = \frac{243499}{163975} = 1.48$$

FIGURE A-11 MIDDLE WALL - LOWER GATE MONOLITH (360' LOCK) M-20

WES FORM NO. 1253A DECEMBER 1964

SLIDING

CASE I NORMAL OPERATIONS

$$R = \Sigma F_v \tan\phi + \text{Key Resistance}$$
$$= (7629.5)(.5658) + 0$$
$$= 4316.8$$

$$SSF = 4316.8 / 1830.2$$
$$= 2.36$$

CASE II MAINTENANCE CONDITION

$$R = (8903.6)(.5658) + 0$$
$$= 5037.7$$

$$SSF = 5037.7 / 4160.0$$
$$= 1.21$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{7629.5}{(35)(40.19)} + \frac{(7629.5)(17.5-13.18)(17.5)(12)}{(40.19)(35)^3}$$
$$= 5.42 + 4.02$$
$$= 9.44 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = \left[\frac{2}{3}\frac{P}{e}\right] \div L$$
$$= \left[\frac{(2)(8903.6)}{(3)(8\ \ )}\right] / 40.19$$
$$= 16.53 \text{ KSF}$$

FIGURE A-11 MIDDLE WALL - LOWER GATE MONOLITH (360' LOCK) M-20

SUBJECT

MIDDLE WALL - LOWER GATE MONOLITH M-22

COMPUTED BY | DATE

CHECKED BY | DATE

STATION 2+81.05 TO 2+36.05

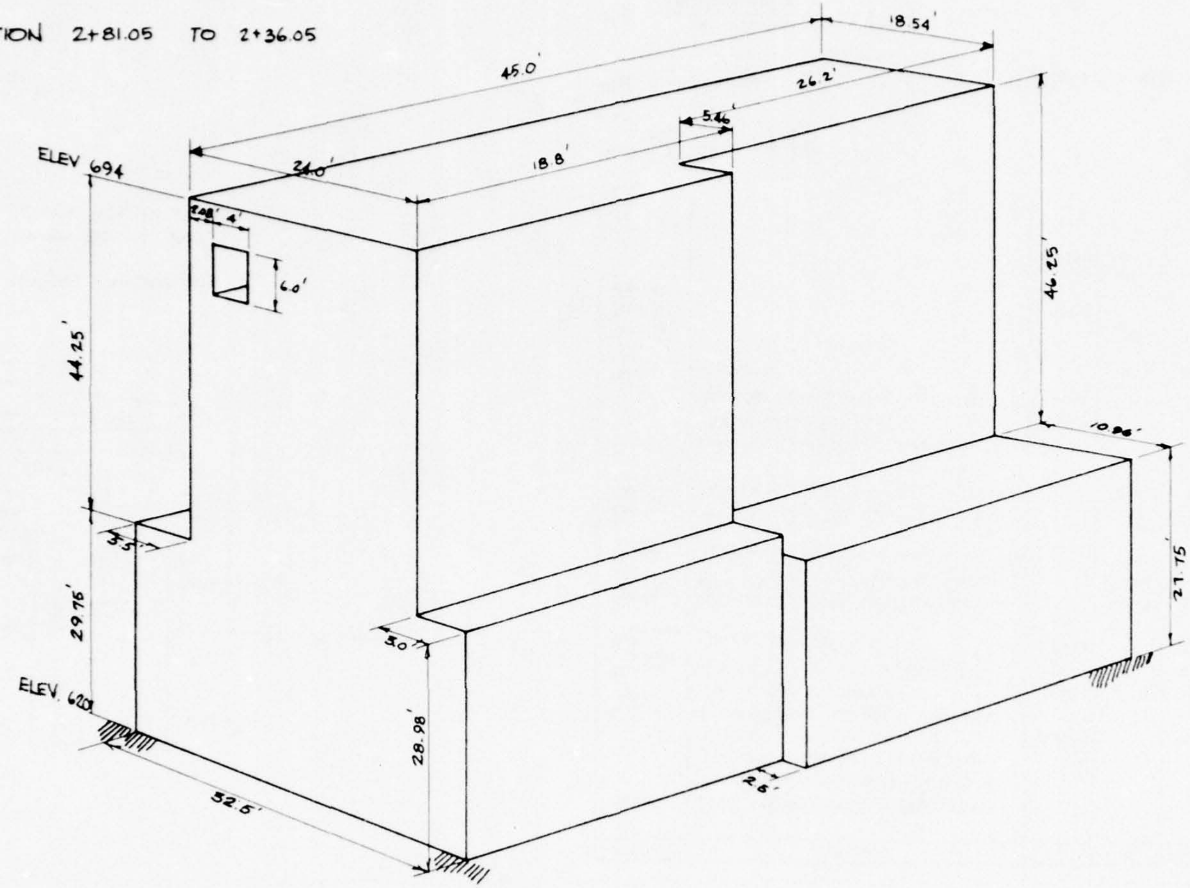


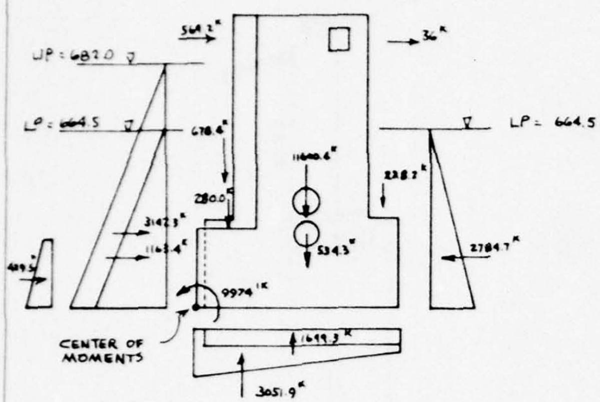
FIGURE A-12 MIDDLE WALL - LOWER GATE MONOLITH M-22

SUBJECT

MIDDLE WALL - LOWER GATE MONOLITH M-22

COMPUTED BY | DATE
CHECKED BY | DATE

NOT TO SCALE



CASE I: NORMAL OPERATIONS

UPPER POOL IN LOCK
LOWER POOL IN RIVER

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	$[.1513((24)(45)(74) + (29.75)(45)(5.5) + (10.96-5.46)(27.75)(26.2) + (648.98-620)(3)(18.8) - (694-647.75)(26.2)(5.46) - (2)(\pi)(5.5)^2(45) - (4)(6)(45))]$	11600.4		18.25	211707
$W_{WATER\ IN\ LOCK}$	$[.0625]((682-647.75)(26.2)(10.96) + (664.5-648.98)(18.8)(3))$	678.4		5.36	3636
$W_{WATER\ IN\ RIVER}$	$[.0625](664.5-649.75)(45)(5.5)$	228.2		32.25	7359
$W_{WATER\ IN\ CULVERTS}$	$[.0625](2)(5.5)^2(\pi)(45)$	534.3		20.25	10820
$P_{WATER\ IN\ LOCK}$	ABOVE GATE $(1/2)(682-620)^2(26.2)(.0625)$		3147.3	20.67	65055
	BELOW GATE $(1/2)(664.5-620)^2(18.8)(.0625)$		1163.4	14.83	17253
$P_{WATER\ IN\ RIVER}$	$(1/2)(664.5-620)^2(45)(.0625)$		-2784.7	14.83	-41297
$P_{EARTH\ \&\ CONC\ IN\ LOCK}$	$(647.75-645.75)(26.2)(.1513)(.5)(645.75-620) + 1/2(645.75-620)^2(26.2)(.5)(.0754)$		429.5	8.58	3685
Uplift	ABOVE GATE $[.0625]((664.5-620)(35)(26.2) + (1/2)(682-664.5)(35)(26.2))$	-3051.9		16.54	-50478
	BELOW GATE $[.0625]((664.5-620)(32.5)(18.8))$	-1699.3		18.75	-31862
HAWSER PULL	$(.8)(45)$		36.0	49.50	1782
GATE S			569.2	72.67	41364
GATE V		280.0		8.12	2274
GATE M					-9974
		8570.1	2560.7		231324

$$e = \frac{231324}{8570.1} = 26.99'$$

PERCENT ACTIVE BASE $\frac{(35-26.99)(3)(45)(100)}{(35)(26.2)+(32.5)(18.8)} = 70.77\%$ ACTIVE BASE

$$F.S. = \frac{\Sigma \text{ STABILIZING M.}}{\Sigma \text{ OVERTURNING M.}} = \frac{281496}{213091} = 1.32$$

FIGURE A-12 MIDDLE WALL LOWER GATE MONOLITH M-22

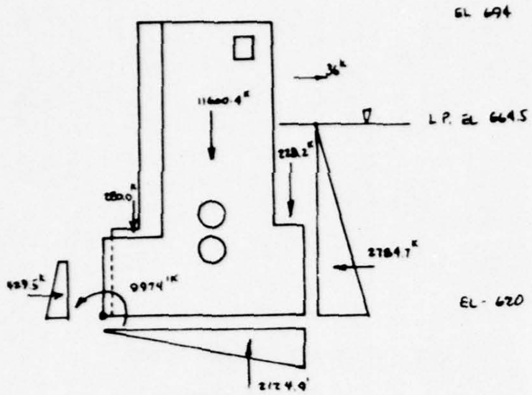
WES FORM NO. 1253A DECEMBER 1966

SUBJECT

MIDDLE WALL - LOWER GATE MONOLITH M-22

COMPUTED BY | DATE
CHECKED BY | DATE

NOT TO SCALE



CASE II MAINTENANCE CONDITION

LOWER POOL IN RIVER
LOCK DEWATERED

ITEM		F_v	F_H	ARM	MOMENT
W_{conc}		11600.4		18.25	211707
$W_{water\ in\ river}$		228.2		32.25	7359
$P_{water\ in\ river}$			-2784.7	14.83	-41297
Uplift	$[.0625][(1/2)(664.5-620)(35)(26.2)+(1/2)(664.5-620)(32.5)(18.8)]$	-2124.9		22.67	-48171
Hawser			36.0	49.50	1782
Gate V		280.0		8.12	2274
Gate M					-9974
$P_{earth\ \&\ conc\ in\ sole}$			429.5	8.58	3685
		9983.7	-2319.2		127365

$$e = \frac{127365}{9983.7} = 12.75$$

PERCENT ACTIVE BASE = 100% because resultant acts within the middle third

$$F.S. = \frac{\sum STAB.\ M}{\sum OVERT.\ M} = \frac{226807}{99442} = 2.28$$

FIGURE A-12 MIDDLE WALL - LOWER GATE MONOLITH M-22

SUBJECT: MIDDLE WALL - LOWER GATE MONOLITH M-22

COMPUTED BY | DATE
CHECKED BY | DATE

SLIDING

CASE I NORMAL OPERATIONS

$$R = \sum F_v \tan\phi + \text{KEY RESISTANCE}$$
$$= 8570.1(.5658) + 0$$
$$= 4848.9$$

$$SSF = 4848.9/2560.7 = 1.89$$

CASE II MAINTENANCE CONDITION

$$R = 9983.7(.5658) + 0$$
$$= 5648.8$$

$$SSF = 5648.8/2319.2 = 2.44$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$e = 35 - 26.99 = 8.01$$

$$f = \frac{2}{3}\frac{P}{e}$$

$$= \frac{(2)(8570.1)}{(3)(8.01)} \div 45 = 15.85 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{P}{A} + \frac{Mc}{I}$$

$$= \frac{9983.7}{(32.5)(18.8)+(35)(26.2)} + \frac{(9983.7)(17.5-12.75)(17.5)}{160781.2}$$

$$= 6.53 + 5.16$$
$$= 11.69 \text{ KSF}$$

FIGURE A-12 MIDDLE WALL LOWER GATE MONOLITH M-22

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: RIVER WALL- UPPER GUARD WALL MONOLITH R-6

COMPUTED BY: | DATE:
CHECKED BY: | DATE:

STATION 4+88.23 A TO 4+4 23 A

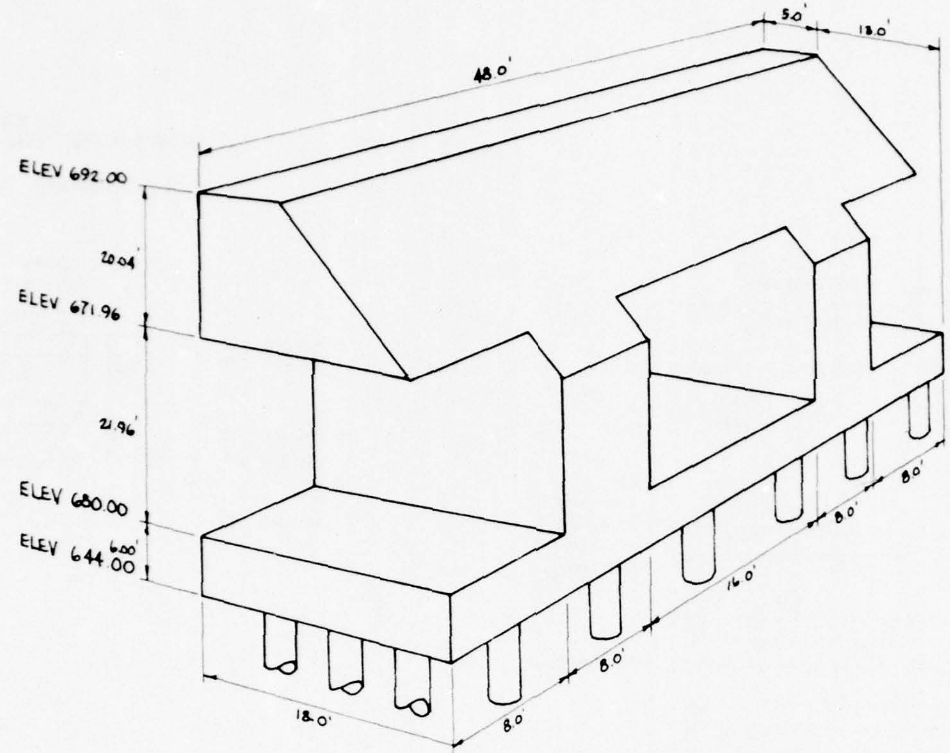


FIGURE A-13 RIVER WALL- UPPER GUARD WALL MONOLITH R-6

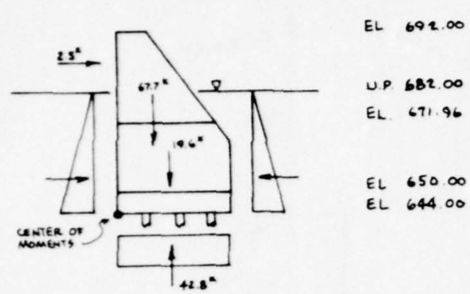
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: RIVER WALL - UPPER GUARD WALL MONOLITH R-6

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



NORMAL OPERATIONS

UPPER POOL BOTH SIDES OF MONOLITH

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$	[.1513][(18)(48)(48) - (1/2)(18)(24)(48) - (18)(18)(16)(2) - (3.96)(15.85)(16)(2) - (1/2)(3.96)(2.15)(16)(2)]/48	67.7		7.42	502
W_{WATER}	[.0625][(21.96)(15.85)(32) + (2.15)(682-650)(32) + (16)(2.15)(682 - 668 - 1.96) + (1/2)(682-671.96)(48)(5.44)]/48	19.6		10.01	196
$P_{WATER\ RIVER}$ $P_{WATER\ LOCK}$	HORIZONTAL WATER FORCES CANCEL				
UPLIFT	[.0625](682-644)(18)	-42.8		9.00	-385
IMPACT	2.5 KIPS/FT		2.50	43.00	108
		44.5	2.50		421

$$e = \frac{421}{44.5} = 9.46$$

PERCENT ACTIVE BASE $\frac{(3)(9.46)(100)}{18} = 100\%$

$$S.F. = \frac{\sum \text{STABILIZING M.}}{\sum \text{OVERTURNING M.}} = \frac{873}{493} = 1.77$$

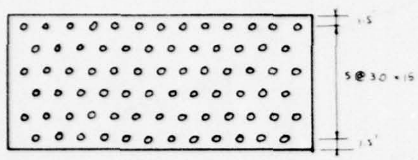
FIGURE A-13 RIVER WALL - UPPER GUARD WALL MONOLITH R-6

SUBJECT: RIVER WALL - UPPER GUARD WALL MONOLITH R-6

COMPUTED BY: DATE:

CHECKED BY: DATE:

HORIZONTAL PILE LOADS



ALLOWABLE HORIZONTAL LOAD PER PILE = 8^K

NORMAL OPERATIONS

$$F_H = \frac{25(48)}{75} = 1.6 \ ^{KIPS}/_{PILE} < 8^K$$

BASE PILE PRESSURE

$I_{\text{pile group}} = 1540.28 \ FT^4$

NORMAL OPERATIONS

$$f = \frac{P}{A} + \frac{Mc}{I}$$

$$= \frac{(44.5)(48)}{.7854(75)} + \frac{(44.5)\left(\frac{18}{2} - 9.46\right)\left(\frac{18}{2}\right)}{1540.28}$$

$$= 36.26 + 0.12$$

$$= 36.38 \text{ KSF}$$

36.38 KSF x .7854 SF/pile = 28.57 Kips/pile < 100 K/pile allowable

FIGURE A-13 RIVER WALL - UPPER GUARD WALL MONOLITH R-6

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: RIVER WALL - UPPER GATE MONOLITH R-12

COMPUTED BY | DATE

CHECKED BY | DATE

STATION 2+53.0A TO 2+11.95A

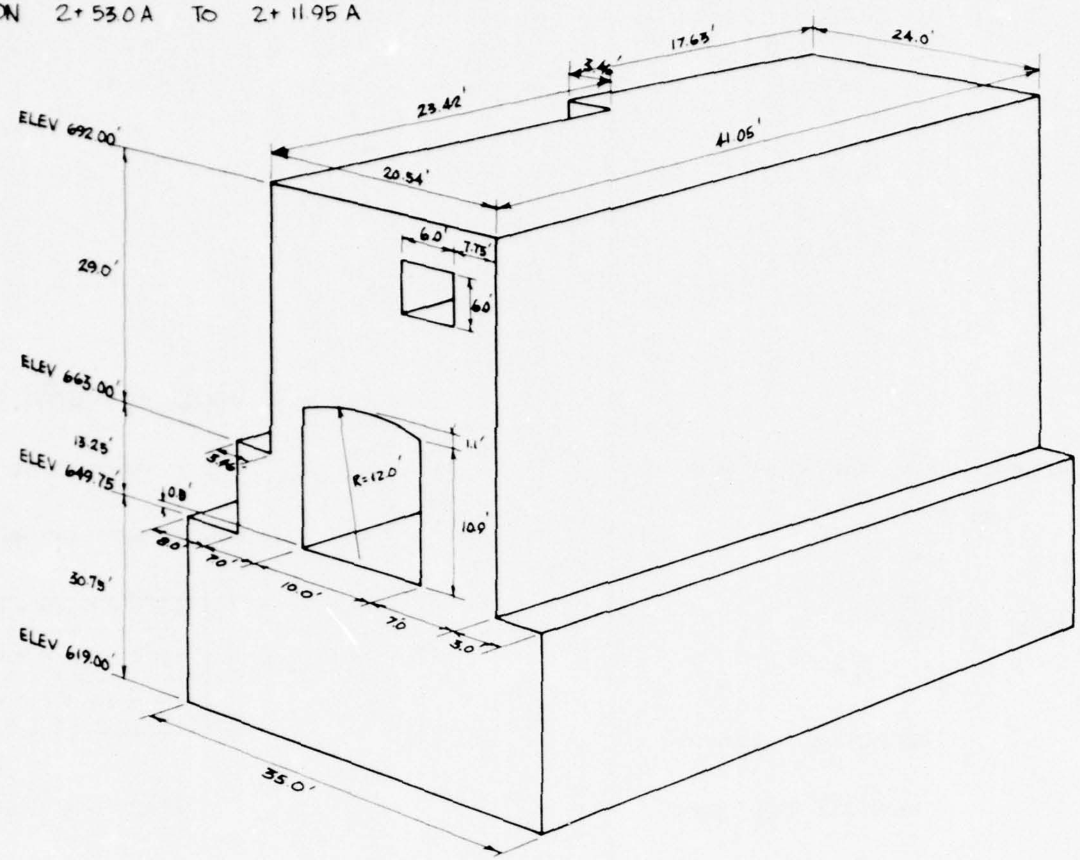


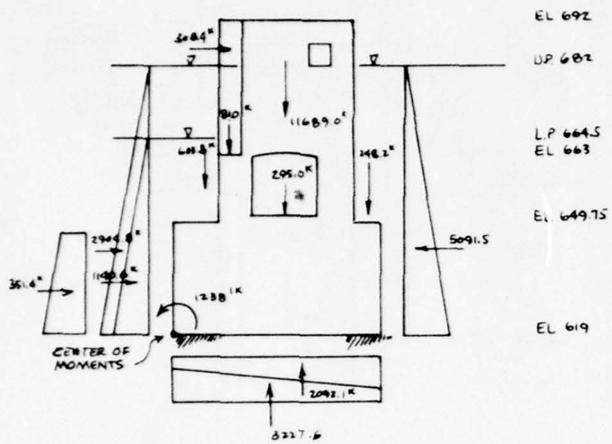
FIGURE A-14 RIVER WALL - UPPER GATE MONOLITH R-12

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: RIVER WALL - UPPER GATE MONOLITH R-12

COMPUTED BY: DATE:
CHECKED BY: DATE:

NOT TO SCALE



CASE I NORMAL OPERATIONS

UPPER POOL ABOVE LOCK
LOWER POOL IN LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513][(3)(30.75)(41.05) + (73)(24)(41.05) + (8)(30.75)(41.05) - (3.46)(23.42)(29) - (6)(6)(41.05) - (10)(11.5)(41.05)]	11689.0		18.80	219753
$W_{WATER, RIVER}$	[.0625](682-649.75)(3)(41.05)	248.2		33.50	8315
$W_{WATER, LOCK}$	[.0625][(8)(17.63)(664.5-649.75) + (8)(682-649.75)(23.42) + (3.46)(682-663)(23.42)]	603.8		4.91	2965
$W_{WATER, CULVERT}$	[.0625](10)(11.5)(41.05)	295.0		20.00	5900
$P_{WATER, RIVER}$	[.0625](1/2)(682-619)²(41.05)		-5091.5	21.00	-106922
$P_{WATER, LOCK}$ (ABOVE GATE)	[.0625](1/2)(682-619)²(23.42)		2904.8	21.00	61001
(IN LOCK)	[.0625](1/2)(664.5-619)²(17.63)		1140.6	15.17	17303
$P_{EARTH \& CONC}$	[.1513](649.75-647.75)(.5)(647.75-619)(17.63) + [.0754](647.75-619)²(.5)(1/2)(17.63)		351.4	10.63	3735
UPLIFT (ABOVE GATE)	[.0625](682-619)(35)(23.42)	-3227.6		17.50	-56483
(IN LOCK)	[.0625][(664.5-619)(35)(17.63) + (1/2)(682-664.5)(35)(17.63)]	-2092.1		16.56	-34646
GATE V		81.0		9.73	788
GATE S			308.4	51.67	15935
GATE M					-1238
		7597.3	-386.3		136451

$$e = \frac{136451}{7597.3} = 17.96$$

$$\text{PERCENT ACTIVE BASE} = \frac{(17.96)(3)(100)}{35} = 100\%$$

$$F.S. = \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{335695}{199244} = 1.68$$

FIGURE A-14 RIVER WALL - UPPER GATE MONOLITH R-12

SUBJECT

RIVER WALL UPPER GATE MONOLITH R-12

COMPUTED BY:	DATE:
CHECKED BY:	DATE:

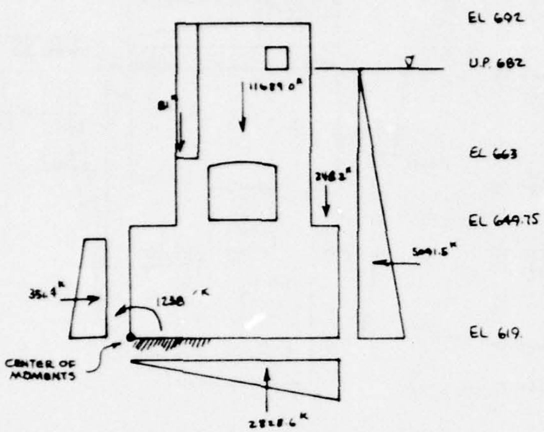
NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$		11689.0		18.80	219753
$W_{WATER, RIVER}$		248.2		33.50	8315
$P_{WATER, RIVER}$			-5091.5	21.00	-106922
$P_{EARTH \& CONC.}$			351.4	10.63	3735
UPLIFT	[.0625](682-619)(35)(1/2)(41.05)	-2828.6		23.33	-65991
GATE V		81.0		9.73	788
GATE M					-1238
		9189.6	-4740.1		58440

$$e = \frac{58440}{9189.6} = 6.36$$

$$\text{PERCENT ACTIVE BASE} = \frac{(6.36)(3)(100)}{(35)} = 54.51\ \%$$

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{232591}{174151} = 1.34$$



CASE II MAINTENANCE CONDITION

UPPER POOL IN RIVER
LOCK DEWATERED

FIGURE A-14 RIVER WALL - UPPER GATE MONOLITH R-12

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: RIVER WALL - UPPER GATE MONOLITH R-12

COMPUTED BY | DATE
CHECKED BY | DATE

SLIDING

CASE I NORMAL OPERATIONS

$R = \sum F_v \tan\phi + \text{Key Resistance}$
$= (7597.3)(.5658) + 0$
$= 4298.6$

$SSF = 4298.6/386.3$
$= 11.1$

CASE II MAINTENANCE CONDITION

$R = (9189.6)(.5658) + 0$
$= 5199.5$

$SSF = 5199.5/4740.1$
$= 1.1$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$f = \frac{P}{A} + \frac{Mc}{I}$
$= \frac{7597.3}{(35)(41.05)} + \frac{(7597.3)(17.96-17.5)(17.5)(12)}{(41.05)(35)^3}$
$= 5.29 \quad 0.42$
$= 5.71$ KSF

CASE II MAINTENANCE CONDITION

$f = \frac{2}{3}\frac{P}{e}/L$
$= \frac{(2)(9189.6)}{(3)(6.36)}/41.05$
$= 23.47$ KSF

FIGURE A-14 RIVER WALL - UPPER GATE MONOLITH R-12

SUBJECT

RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-13

COMPUTED BY	DATE
CHECKED BY	DATE

STATION 2+11.95A TO 1+68.0A

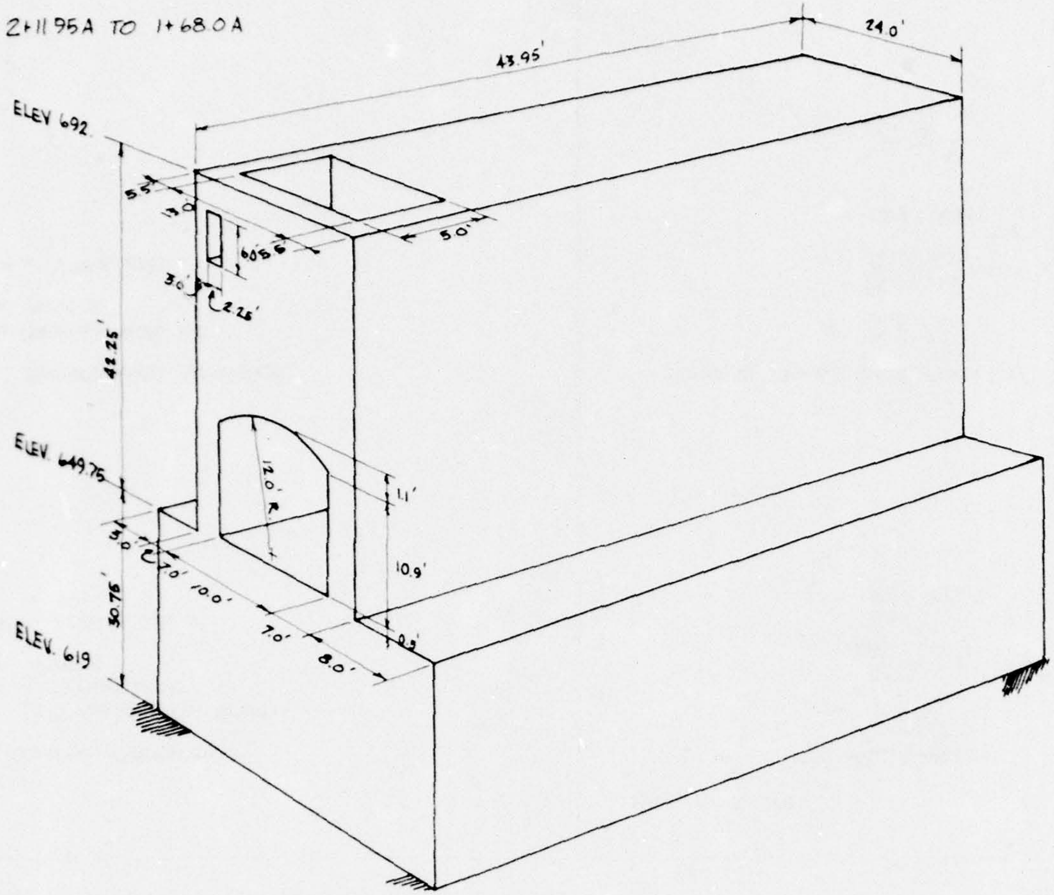


FIGURE A-15 RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-13

WES FORM NO. 1253A DECEMBER 1964

SUBJECT

RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-13

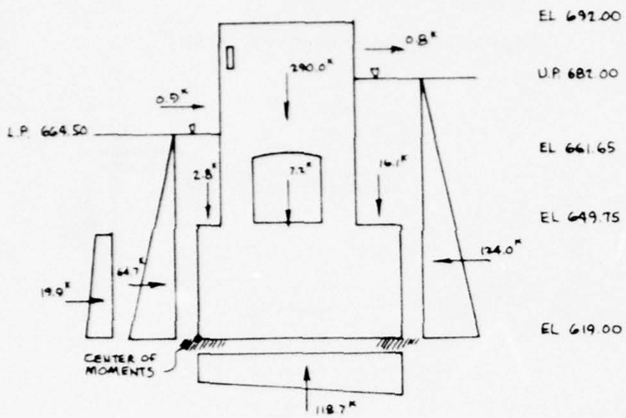
COMPUTED BY

CHECKED BY

DATE

DATE

NOT TO SCALE



CASE I NORMAL OPERATIONS

UPPER POOL IN RIVER
LOWER POOL IN LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513][(24)(73) + (8)(30.75) + (3)(30.75) - (10)(11.5) - (6)(2.25) - (13)(5)(692 - 661.65)/43.95]	290.0		16.46	4773
W_{WATER} IN RIVER	[.0625](8)(682-649.75)	16.1		31.00	499
W_{WATER} IN LOCK	[.0625](3)(664.5-649.75)	2.8		1.50	4
W_{WATER} IN CULVERT	[.0625](10)(11.5)	7.2		15.00	108
$P_{CONC \& EARTH}$	[.1513](649.75-647.75)(.5)(647.75-619)+ [.0754](647.75-619)²(1/2)(.5)		19.9	10.62	211
P_{WATER} IN RIVER	[.0625](1/2)(682-619)²		-124.0	21.00	-2604
P_{WATER} IN LOCK	[.0625](1/2)(664.5-619)²		64.7	15.17	981
Uplift	[.0625][(664.5-619)(35) + (1/2)(35)(682-664.5)]	-118.7		18.44	-2189
IMPACT	0.9 KIPS/FT		0.9	50.50	45
HAWSER	0.8 KIPS/FT		0.8	68.00	54
		197.4	-37.7		1882

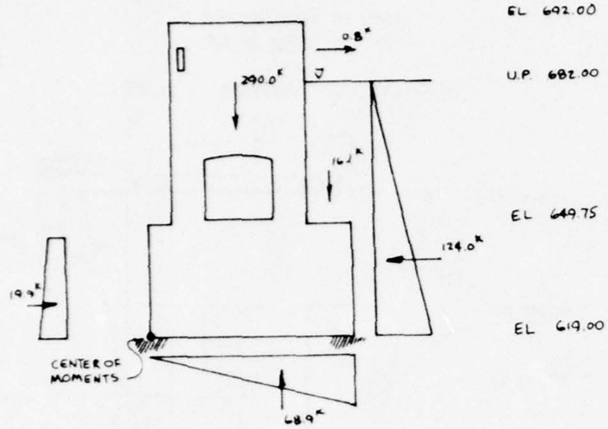
$$e = \frac{1882}{197.4} = 9.53'$$

$$\text{PERCENT ACTIVE BASE} = \frac{9.53(3)(100)}{35} = 81.69\%$$

$$F.S. = \frac{\Sigma \text{ STABILIZING M.}}{\Sigma \text{ OVERTURNING M}} = \frac{6675}{4793} = 1.39$$

FIGURE A-15 RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-13

NOT TO SCALE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		290.0		16.46	4773
$W_{WATER\ IN\ RIVER}$		16.1		31.00	499
$P_{WATER\ IN\ RIVER}$			-124.0	21.00	-2604
$P_{CONC\ \&\ EARTH}$			19.9	10.62	211
UPLIFT	[.0625][682-619](35)	-68.9		23.33	-1607
HAWSER			0.8	68.00	54
		237.2	-103.3		1326

$$e = \frac{1326}{237.2} = 5.59'$$

$$\text{PERCENT ACTIVE BASE} = \frac{5.59(3)(100)}{35} = 47.91\%$$

$$F.S. = \frac{\Sigma \text{ STAB M}}{\Sigma \text{ OVERT. M}} = \frac{5537}{4211} = 1.31$$

CASE II MAINTENANCE CONDITION

UPPER POOL IN RIVER
LOCK DEWATERED

FIGURE A-15 RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-13

SUBJECT: RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-13

COMPUTED BY: | DATE:
CHECKED BY: | DATE:

SLIDING

CASE I NORMAL OPERATIONS

$R = \Sigma F_v \tan\phi$ + KEY RESISTANCE
$= (197.4)(.5658) + 0$
$= 111.7$

$SSF = 111.7/37.7 = 2.96$

CASE II MAINTENANCE CONDITION

$R = (237.2)(.5658) + 0$
$= 134.2$

$SSF = 134.2/103.3 = 1.30$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \frac{2}{3}\frac{P}{e} = \frac{(2)(197.4)}{(3)(9.53)} = 13.81 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{2}{3}\frac{P}{e} = \frac{(2)(237.2)}{(3)(6.59)} = 28.29 \text{ KSF}$$

FIGURE A-15 RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-13

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-15

COMPUTED BY: | DATE:
CHECKED BY: | DATE:

STATION 1+22.67A TO 0+91.35A

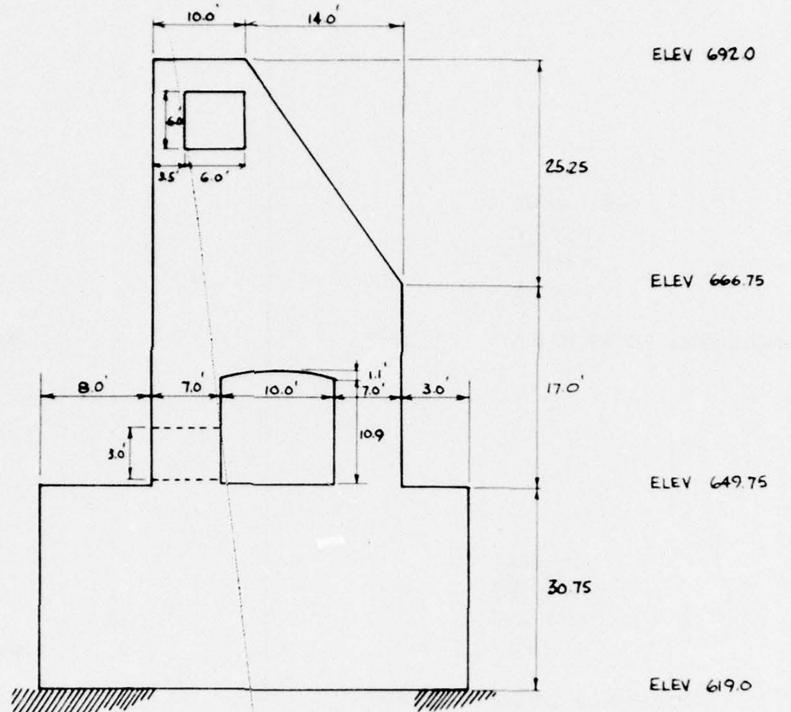


FIGURE A-16 RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-15

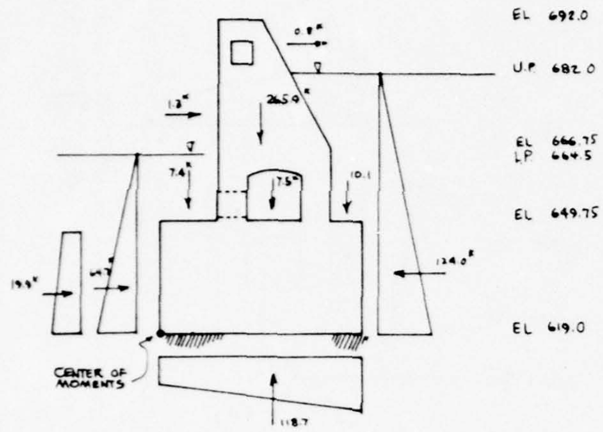
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-15

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE I NORMAL OPERATIONS

UPPER POOL IN RIVER
LOWER POOL IN LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{conc}	$[.1513][(42.25)(24)+(30.75)(35)-(\frac{1}{2})(692-666.75)(14)-(6)(6)-(10)(11.5)-(3)(4)(7)(2)/31.32]$	265.9		17.87	4752
$W_{water, river}$	$[.0625][(682-649.75)(3)+(\frac{1}{2})(8.46)(682-666.75)]$	10.1		31.77	321
$W_{water, lock}$	$[.0625](664.5-649.75)(8)$	7.4		4.00	30
$W_{water\ in\ culverts}$	$[.0625][(10)(11.5)+(3)(4)(7)(2)/31.32]$	7.5		19.61	147
$P_{water, river}$	$[.0625](682-619)^2(\frac{1}{2})$		-124.0	21.00	-2604
$P_{water, lock}$	$[.0625](664.5-619)^2(\frac{1}{2})$		64.7	15.17	981
$P_{earth\ \&\ conc\ in\ lock}$	$[.1515](649.75-647.75)(.5)(647.75-619)+[.0754](647.75-619)^2(\frac{1}{2})(.5)$		19.9	10.63	212
Uplift	$[.0625][(664.5-619)(35)+(\frac{1}{2})(35)(682-664.5)]$	-118.7		18.44	-2189
Impact			1.3	50.50	66
Hawser			0.8	68.00	54
		172.2	-37.3		1770

$$e = \frac{1770}{172.2} = 10.28'$$

$$\text{PERCENT ACTIVE BASE} = \frac{(10.28)(3)(100)}{35} = 88.11\%$$

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{6563}{4793} = 1.37$$

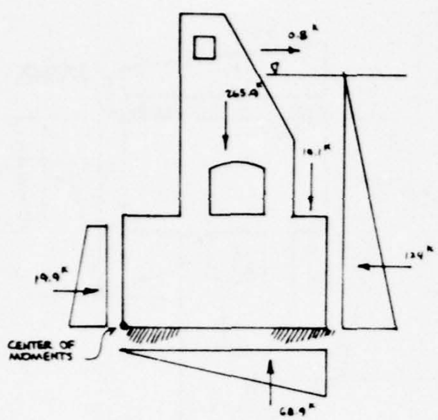
FIGURE A-16 RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-15

SUBJECT RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-15

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE II MAINTENANCE CONDITION

UPPER POOL IN RIVER
LOCK DEWATERED

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$		265.9		17.87	4752
$W_{WATER\ RIVER}$		10.1		31.77	321
$P_{WATER,\ RIVER}$			-124.0	21.00	-2604
$P_{EARTH\ \&\ CONC}$			19.9	10.63	212
UPLIFT	$[.0625](682-619)(1/2)(35)$	-68.9		23.33	-1607
HAWSER			0.8	68.00	54
		207.1	-103.3		1128

$$e = \frac{1128}{207.1} = 5.45$$

$$\text{PERCENT ACTIVE BASE} = \frac{(5.45)(3)(100)}{35} = 46.71\%$$

$$F.S. \ \frac{\sum \text{STABILIZING M}}{\sum \text{OVERTURNING M}} = \frac{5339}{4211} = 1.27$$

FIGURE A-16 RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-15

WES FORM NO. 1253A DECEMBER 1964

SLIDING

CASE I NORMAL OPERATIONS

$$R = \sum F_v \tan\phi + \text{KEY RESISTANCE}$$
$$= (172.2)(.5658) + 0$$
$$= 97.4$$

$$SSF = 97.4/37.3$$
$$= 2.61$$

CASE II MAINTENANCE CONDITION

$$R = (207.1)(.5658) + 0$$
$$= 117.2$$

$$SSF = 117.2/103.3$$
$$= 1.13$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \frac{2}{3}\frac{P}{e}$$
$$= \frac{(2)(172.2)}{(3)(10.28)}$$
$$= 11.17 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{2}{3}\frac{P}{e}$$
$$= \frac{(2)(207.1)}{(3)(5.45)}$$
$$= 25.33 \text{ KSF}$$

FIGURE A-16 RIVER WALL - TYPICAL UPPER CHAMBER MONOLITH R-15

SUBJECT: RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-20

COMPUTED BY: DATE:

CHECKED BY: DATE:

STATION 0+94.0B TO 0+47.45 B

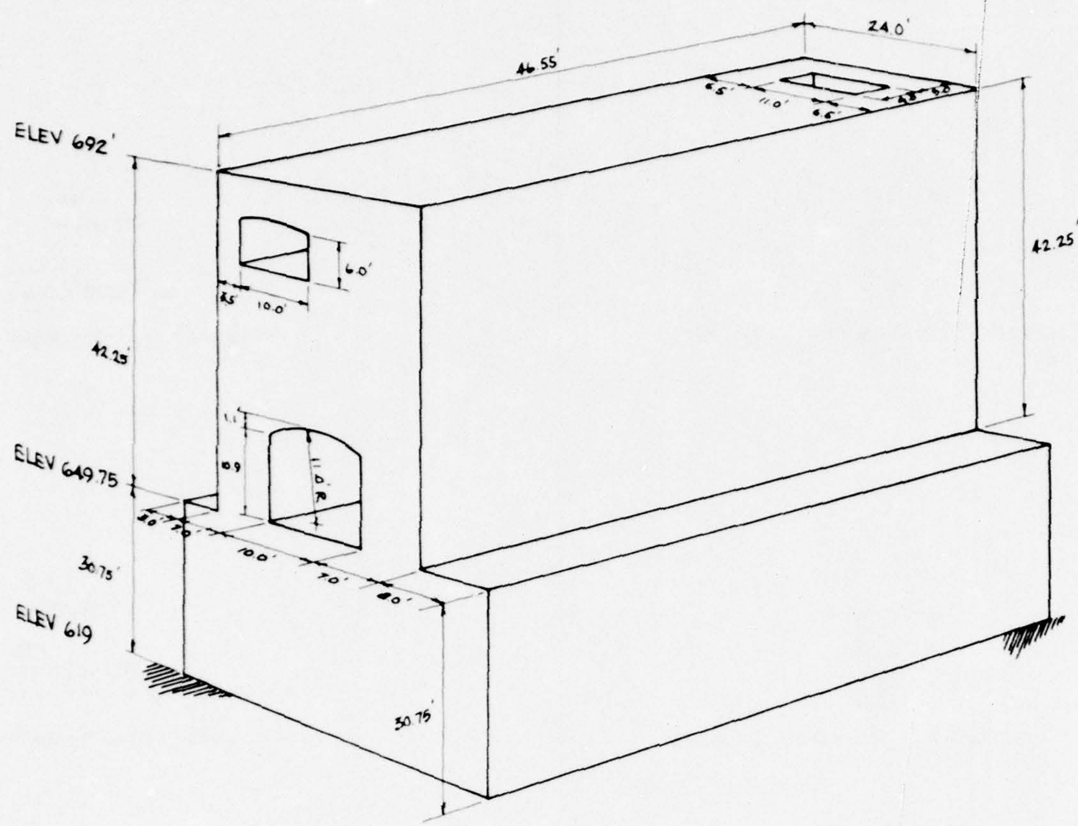


FIGURE A-17 RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-20

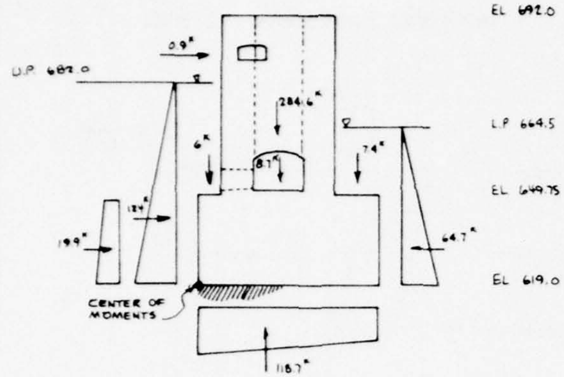
WES FORM NO. 1253A

SUBJECT: RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-20

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE I: NORMAL OPERATIONS

UPPER POOL IN LOCK
LOWER POOL IN RIVER

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513][(42.25)(24) + (35)(30.75) - (6)(10) - (10)(11.5) - (3)(4)(7)(3)/46.55 - (4)(11)(30.35)/46.55]	284.6		16.62	4730
$W_{WATER, RIVER}$	[.0625](664.5 - 649.75)(8)	7.4		31.00	229
$W_{WATER, LOCK}$	[.0625](682 - 649.75)(3)	6.0		1.50	9
$W_{WATER\ IN\ CULVERTS}$	[.0625][(10)(11.5) + (3)(4)(7)(3)/46.55 + (682 - 661.65)(4)(11)/46.55]	8.7		14.67	128
$P_{WATER, RIVER}$	[.0625](664.5 - 619)^2(1/2)		-64.7	15.17	-981
$P_{WATER, LOCK}$	[.0625](682 - 619)^2(1/2)		124.0	21.00	2604
$P_{EARTH\ \&\ CONC\ IN\ LOCK}$	[.1513](649.75 - 647.75)(.5)(647.75 - 619) + [.0754](647.75 - 619)^2(.5)(1/2)		19.9	10.63	212
UPLIFT	[.0625][(664.5 - 619)(35) + (682 - 664.5)(1/2)(35)]	-118.7		16.56	-1965
IMPACT			0.9	68.00	61
		188	80.1		5027

$$e = \frac{5027}{188} = 26.74$$

PERCENT ACTIVE BASE $\frac{(35 - 26.74)(3)(100)}{35} = 70.8$

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{6619}{5066} = 1.31$$

FIGURE A-17 RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-20

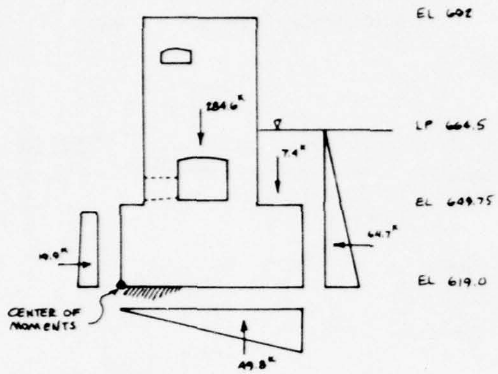
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-20

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE II MAINTENANCE CONDITION

LOWER POOL IN RIVER
LOCK DEWATERED

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		284.6		16.62	4730
$W_{WATER-RIVER}$		7.4		31.00	229
$P_{WATER-RIVER}$			-64.7	15.17	-981
$P_{EARTH+CONC\ IN\ LOCK}$			19.9	10.65	212
UPLIFT	$[0.625](1/2)(664.5-619)(35)$	-49.8		23.33	-1161
		242.2	-44.8		3029

$e = \frac{3029}{242.2} = 12.51$

PERCENT ACTIVE BASE $= \frac{(12.51)(3)(100)}{35} = 100\%$

F.S. $\frac{\sum \text{STABILIZING M}}{\sum \text{OVERTURNING M}} = \frac{5171}{2142} = 2.41$

FIGURE A-17 RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH - R-20

SUBJECT

RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-20

COMPUTED BY | DATE

CHECKED BY | DATE

SLIDING

CASE I NORMAL OPERATION

$$R = \Sigma F_v \tan\phi + \text{KEY RESISTANCE}$$
$$= (188)(.5658) + 0$$
$$= 106.4$$

$$SSF = 106.4/80.1$$
$$= 1.33$$

CASE II MAINTENANCE CONDITION

$$R = (242.2)(.5658) + 0$$
$$= 137.0$$

$$SSF = 137.0/44.8$$
$$= 3.06$$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$$f = \frac{2}{3}\frac{P}{e}$$
$$= \frac{(2)(188)}{(3)(35-26.74)}$$
$$= 15.2 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{242.2}{35} + \frac{(242.2)\left(\frac{35}{2} - 12.51\right)\left(\frac{35}{2}\right)(12)}{35^3}$$
$$= 6.92 + 5.92$$
$$= 12.8 \text{ KSF}$$

FIGURE A-17 RIVER WALL - TYPICAL LOWER CHAMBER MONOLITH R-20

SUBJECT: RIVER WALL - LOWER GATE MONOLITH R-23

COMPUTED BY | DATE

CHECKED BY | DATE

STATION 1+88.04 B TO 1+47.0 B

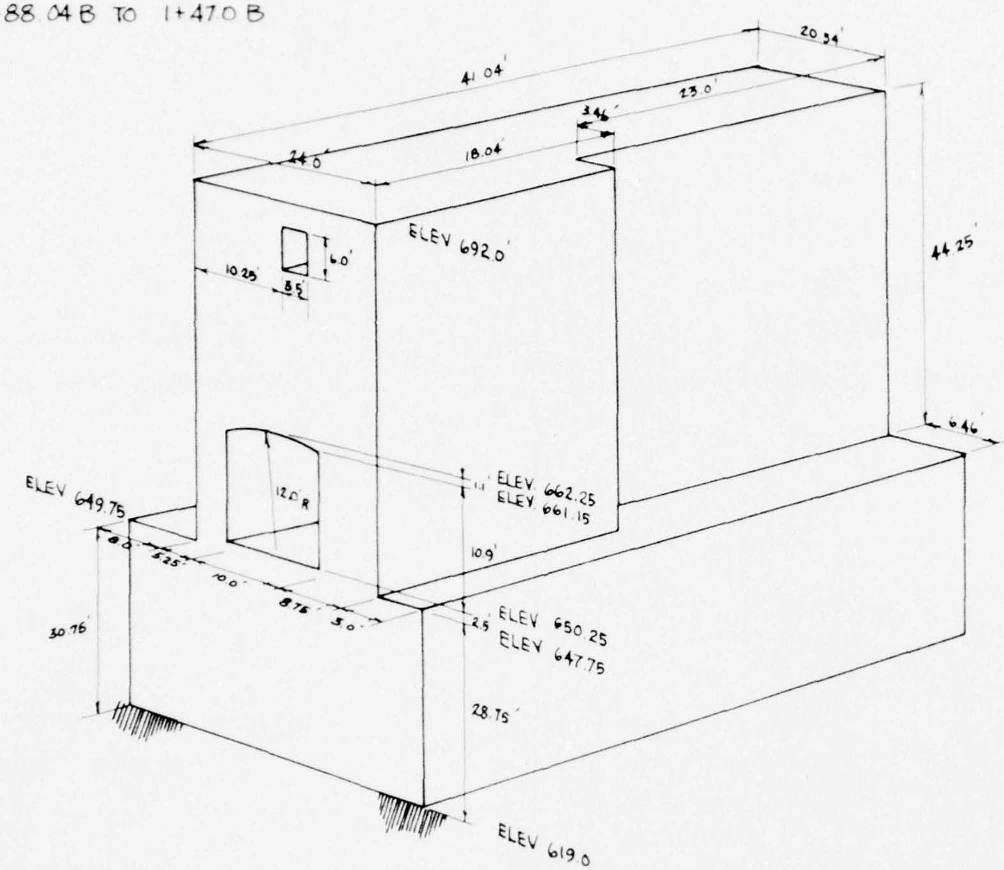


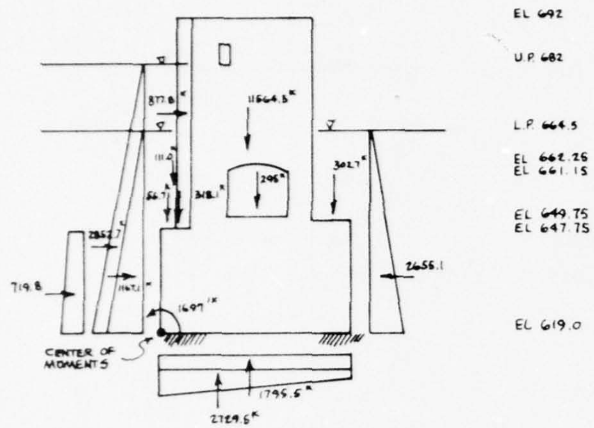
FIGURE A-18 RIVER WALL-LOWER GATE MONOLITH R-23

SUBJECT: RIVER WALL - LOWER GATE MONOLITH R-23

COMPUTED BY | DATE

CHECKED BY | DATE

NOT TO SCALE



CASE I NORMAL OPERATIONS

LOWER POOL IN RIVER
LOWER POOL BELOW LOCK
UPPER POOL IN LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$	[.1513][(24)(41.04)(73) + (8)(41.04)(30.75) + (3)(41.04)(28.75) - (3.46)(23)(44.25) - (10)(11.5)(41.04) - (6)(3.5)(41.04)]	11564.5		16.85	194862
$W_{WATER, RIVER}$	[.0625](664.5 - 649.75)(41.04)(8)	302.7		31.00	9384
$W_{WATER, LOCK}$ (ABOVE GATE)	[.0625](682 - 647.75)(6.46)(23)	318.1		3.23	1027
(BELOW GATE)	[.0625](664.5 - 647.75)(3)(18.04)	56.7		1.50	85
$W_{WATER, CULVERT}$	[.0625](10)(11.5)(41.04)	295.0		16.75	4941
$P_{WATER, RIVER}$	[.0625](664.5 - 619)^2(1/2)(41.04)		-2655.1	21.00	-55757
$P_{WATER, LOCK}$ (ABOVE GATE)	[.0625](682 - 619)^2(1/2)(23)		2852.7	21.00	59907
(BELOW GATE)	[.0625](664.5 - 619)^2(1/2)(18.04)		1167.1	15.17	17705
$P_{EARTH\ \&\ CONC}$	[[.1513](647.75 - 645.75)(.5)(645.75 - 619) + [.0754](645.75 - 619)^2(1/2)(.5)](41.04)		719.8	9.94	7155
UPLIFT (ABOVE GATE)	[.0625][(664.5 - 619)(35)(23) + (1/2)(682 - 664.5)(35)(23)]	-2729.5		16.56	-45200
(BELOW GATE)	[.0625](664.5 - 619)(35)(18.04)	-1795.5		17.50	-31421
GATE V		111.0		4.29	476
GATE S			877.8	43.05	37789
GATE M					-1697
		8123	2962.3		199256

$$e = \frac{199256}{8123} = 24.53'$$

$$\text{PERCENT ACTIVE BASE} = \frac{(35.00 - 24.53)(3)(100)}{35} = 89.74\%$$

$$F.S. = \frac{\Sigma \text{ STABILIZING } M}{\Sigma \text{ OVERTURNING } M} = \frac{289338.6}{254309.2} = 1.14$$

FIGURE A-18 RIVER WALL - LOWER GATE MONOLITH R-23

SUBJECT: RIVER WALL - LOWER GATE MONOLITH R-23

COMPUTED BY: DATE:

CHECKED BY: DATE:

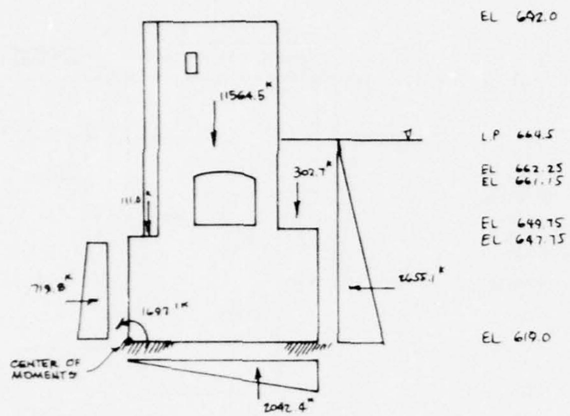
NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
$W_{CONC.}$		11564.5		16.85	194862
$W_{WATER, RIVER}$		302.7		31.00	9384
$P_{WATER, RIVER}$			-2655.1	21.00	-55757
$P_{EARTH \& CONC}$			719.8	9.94	7155
UPLIFT	$[.0625](664.5-619)(35)(41.04)(1/2)$	-2042.4		23.33	-47649
GATE V		111.0		4.29	476
GATE M					-1697
		9935.8	-1935.3		106774

$$e = \frac{106774}{9935.8} = 10.75$$

$$\text{PERCENT ACTIVE BASE} = \frac{(10.75)(3)(100)}{35} = 92.14\%$$

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{211877}{105103} = 2.02$$



CASE II MAINTENANCE CONDITION

LOCK DEWATERED
LOWER POOL IN RIVER

FIGURE A-18 RIVER WALL - LOWER GATE MONOLITH R-23

WES FORM NO. 1253A DECEMBER 1964

SLIDING

CASE I NORMAL OPERATIONS

$R = \sum F_V \tan\phi$ + KEY RESISTANCE

$= (8123)(.5658)$

$= 4596$

$SSF = 4596/2962.3$

$= 1.55$

CASE II MAINTENANCE CONDITION

$R = 9935.8\,(.5658)$

$= 5621.7$

$SSF = 5621.7/1935.3$

$= 2.9$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$f = \left(\frac{2}{3}\right)\left(\frac{P}{e}\right)$

$= \frac{(2)(8123)}{(3)(35-24.53)} \div 41.04$

$= 12.60$ KSF

CASE II MAINTENANCE CONDITION

$f = \left(\frac{2}{3}\right)\left(\frac{P}{e}\right)$

$= \frac{(2)(9935.8)}{(3)(10.75)} \div 41.04$

$= 15.01$ KSF

FIGURE A-18 RIVER WALL - LOWER GATE MONOLITH R-23

SUBJECT: RIVER WALL - LOWER GUARD WALL R-29

COMPUTED BY: DATE:

CHECKED BY: DATE:

STATION 3+98.21 B TO 4+40.21 B

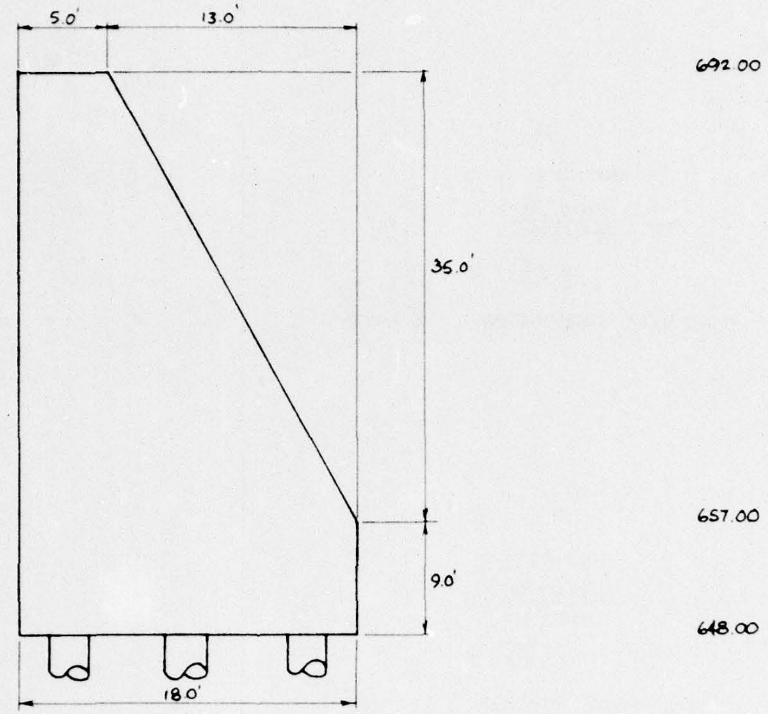


FIGURE A-19 RIVER WALL - LOWER GUARD WALL MONOLITH R-29

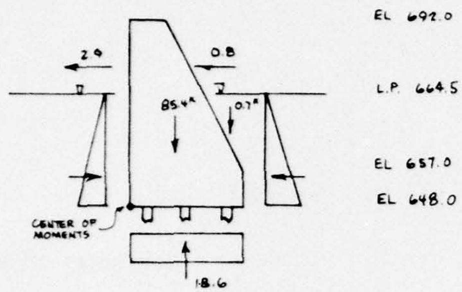
WES FORM NO. 1253A

SUBJECT: RIVER WALL LOWER GUARD WALL R-29

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE I NORMAL OPERATION

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513]((18)(44) - (1/2)(692-657)(15))	85.4		7.12	608
$W_{WATER\ IN\ RIVER}$	[.0625](664.5-657)(27.9)(1/2)	0.7		17.07	12
$P_{WATER\ RIVER}$					
$P_{WATER\ LOCK}$	Horizontal water pressures cancel				
U_{PLIFT}	[.0625](664.5-648)(18)	-18.6		9.00	-167
HAWSER	0.8 KIPS/FT		-0.8	21.50	-17
IMPACT	120/42 = 2.86		-2.9	21.50	-62
		67.5	-3.7		347

$$e = \frac{347}{67.5} = 5.54$$

PERCENT ACTIVE BASE $\frac{5.54(3)(100)}{18} = 92.33\ \%$

$$F.S. = \frac{\sum \text{STABILIZING MOMENTS}}{\sum \text{OVERTURNING MOMENTS}} = \frac{620}{246} = 2.52$$

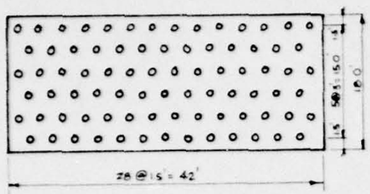
FIGURE A-19 RIVER WALL - LOWER GUARD WALL MONOLITH R-29

SUBJECT: RIVER WALL - LOWER GUARD WALL R-29

COMPUTED BY: DATE:

CHECKED BY: DATE:

HORIZONTAL PILE LOADS



ALLOWABLE HORIZONTAL LOAD PER PILE = 8.0^K

NORMAL OPERATIONS

$$F_H = \frac{3.7(42)}{81} = 1.92^K/\text{pile} < 8^K$$

BASE PILE PRESSURE

NORMAL OPERATIONS

I pile group = 1673.93 FT^4

$$f = \frac{P}{A} + \frac{Mc}{I}$$

$$= \frac{(67.5)(42)}{.7854(81)} + \frac{(67.5)(42)\left(\frac{18}{2} - 5.54\right)\left(\frac{18}{2}\right)}{1673.93}$$

$$= 44.56 + 52.74$$

$$= 97.30 \text{ KSF}$$

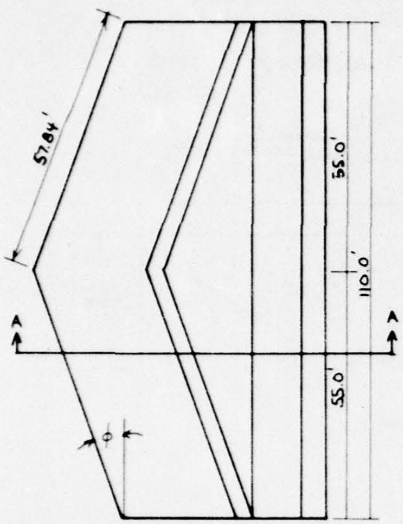
$$97.30 \text{ KSF} \times .7854 \text{ SF/pile} = 76.42 \frac{K}{\text{pile}} < 100 \, {}^{K}/_{\text{pile}}$$

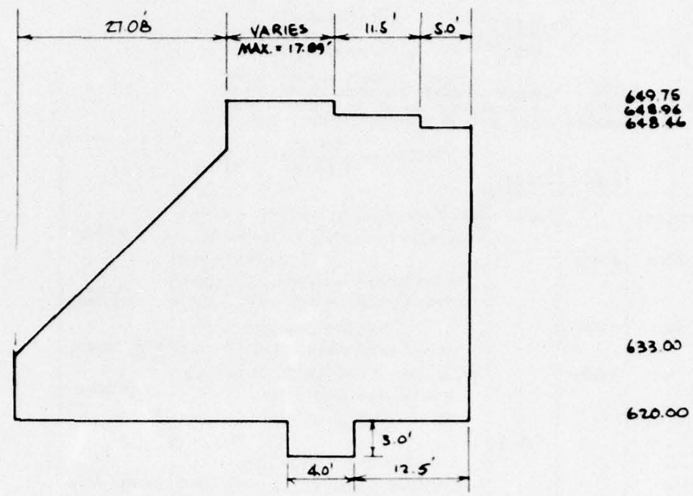
FIGURE A-19 RIVER WALL - LOWER GUARD WALL MONOLITH R-29

WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LOWER MITER SILL (110' LOCK)

COMPUTED BY: | DATE:
CHECKED BY: | DATE:





SECTION AA

FIGURE A-20 LOWER MITER SILL (110' LOCK)

WES FORM NO. 1253A DECEMBER 1964

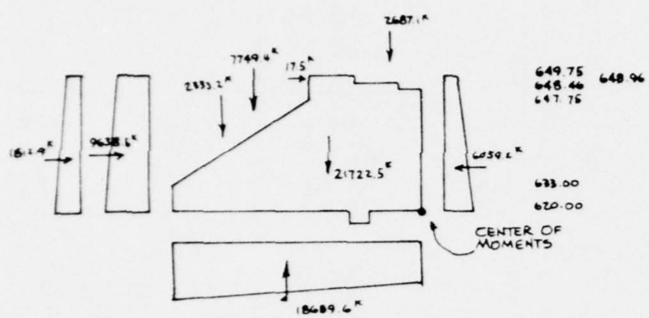
SUBJECT

LOWER MITER SILL (110' LOCK)

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE I NORMAL OPERATIONS

UPPER POOL IN LOCK
LOWER POOL BELOW LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONCR}	[.1513][(648.46-620)(5)(110) + (648.96-620)(11.5)(110) + (2)(1/2)(649.75-620)(17.89)(56) + (2)(1/2)(647.75-633)(25.75)(57.84) + (2)(633-620)(25.75)(57.84) + (4)(3)(110)]	21722.5		-23.52	-510913
$W_{WATER, \text{ IN LOCK}}$	[.0625][(2)(682-647.75)(57.84)(25.75) + (2)(1/2)(647.75-633)(57.84)(25.75)]	7749.4		-39.78	-308271
$W_{WATER, \text{ BELOW LOCK}}$	[.0625][(2)(1/2)(17.89)(55)(664.5-649.75) + (11.5)(110)(664.5-648.96) + (5)(110)(664.5-648.46)]	2687.1		-13.01	-34959
$W_{CONC \& EARTH \text{ IN LOCK}}$	[.1513](647.75-645.75)(57.84)(25.75)(2) + [.0754](2)(1/2)(645.75-633)(25.75)(57.84)	2333.2		-41.75	-97411
$P_{FROM\ GATE, \text{ ON SILL}}$	18.34 (cos φ) = 18.34 (.9509) = 17.50		17.5	29.75	521
$P_{WATER \text{ IN LOCK}}$	[.0625][cos φ][(2)(682-649.75)(644.75-620)(57.84) + (2)(1/2)(649.75-620)²(57.84)]		9638.5	13.31	128288
$P_{WATER, \text{ BELOW LOCK}}$	[.0625][(664.5-649.75)(649.75-620)(110) + (1/2)(649.75-620)²(110)]		-6059.2	12.38	-75013
$P_{EARTH \& CONC \text{ IN LOCK}}$	[cos φ][[.1513](647.75-645.75)(2)(645.75-620)(.5)(57.84) + [.0754](2)(1/2)(645.75-620)²(57.84)(.5)]		1812.9	9.59	17386
U_{PLIFT}	[2][.0625][55][(664.5-620)(43.58) + (1/2)(56.9-44.5)(43.58) + (1/2)(56.9)(17.89)]	-18689.6		-27.72	+518076
		15802.6	5049.7		-362296

$$e = \frac{-362296}{15802.6} = -22.92$$

ANALYSIS SHOWS THAT FOR A BASE OF THESE DIMENSIONS IF THE RESULTANT FALLS WITHIN 7.04' of the CENTER OF GRAVITY THE ENTIRE BASE IS IN COMPRESSION
CG IS 26.52' FROM THE CENTER OF MOMENTS 26.52 - 22.92 = 3.60 < 7.04
PERCENT ACTIVE BASE = 100%

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{1026567}{664271} = 1.55$$

FIGURE A-20 LOWER MITER SILL (110' LOCK)

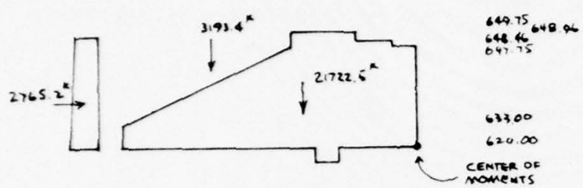
WES FORM NO. 1253A DECEMBER 1964

SUBJECT: LOWER MITER SILL (110' LOCK)

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE II MAINTENANCE CONDITION

LOCK DEWATERED

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}		21722.5		-23.52	-510913
W_{EARTH} & CONC IN LOCK	[1513](647.75 - 645.75)(57.84)(25.75)(2) + [1203](2)(½)(645.75 - 633)(57.84)(25.75)	3193.4		-42.22	-314825
P_{EARTH} & CONC IN LOCK	[1513](2)(647.75-645.75)(.5)(645.75-620)(57.84) + [1207](2)(½)(.5)(645.75-620)²(57.84)		2765.2	9.28	25661
		24915.9	2765.2		-800077

$$e = \frac{-800077}{24915.9} - 32.11$$

ANALYSIS SHOWS THAT FOR A BASE OF THESE DIMENSIONS SUBJECTED TO THIS LOADING IF THE RESULTANT FALLS WITHIN 9.27' FROM THE CENTER OF GRAVITY THE ENTIRE BASE IS IN COMPRESSION. CG IS 26.52' FROM THE CENTER OF MOMENTS 32.11 - 26.52 = 5.59 < 9.27

PERCENTAGE ACTIVE BASE = 100%

$$F.S. = \frac{\sum \text{STABILIZING } M}{\sum \text{OVERTURNING } M} = \frac{825738}{25661} = 32.18$$

FIGURE A-20 LOWER MITER SILL (110' LOCK)

SUBJECT: LOWER MITER SILL (110' LOCK)

COMPUTED BY: DATE:

CHECKED BY: DATE:

SLIDING

CASE I NORMAL OPERATIONS

$$R = \sum F_v \tan\theta + \text{Key Resistance}$$
$$= (15802.6)(.5658) + (4)(110)(.075)(144)$$
$$= 8941.1 + 4752$$
$$= 13693.1$$

$$SSF = 13693.1 / 5049.7$$
$$= 2.71$$

CASE II MAINTENANCE CONDITION

$$R = (24915.9)(.5658) + 4752$$
$$= 14097.4 + 4752$$
$$= 18849.4$$

$$SSF = 18849.4 / 2765.2$$
$$= 6.82$$

BASE PRESSURES

CASE I NORMAL OPERATION

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{15802.6}{5777.8} + \frac{(15802.6)(26.52 - 22.92)(26.52)}{1404872}$$
$$= 2.74 + 1.07$$
$$= 3.81 \text{ KSF}$$

CASE II MAINTENANCE CONDITION

$$f = \frac{P}{A} + \frac{Mc}{I}$$
$$= \frac{24915.9}{5777.8} + \frac{24915.9(32.11 - 26.52)(61.47 - 26.52)}{1,404,872}$$
$$= 4.31 \quad 3.46$$
$$= 7.77 \text{ KSF}$$

NOTE THE ANALYSIS FOR THE UPPER MITER SILL (110' LOCK) WAS FOUND TO BE ADEQUATE AND STABLE AND HAS NOT BEEN PRESENTED HERE.

FIGURE A-20 LOWER MITER SILL (110' LOCK)

SUBJECT	COMPUTED BY	DATE
LOWER MITER SILL (56' LOCK)	CHECKED BY	DATE

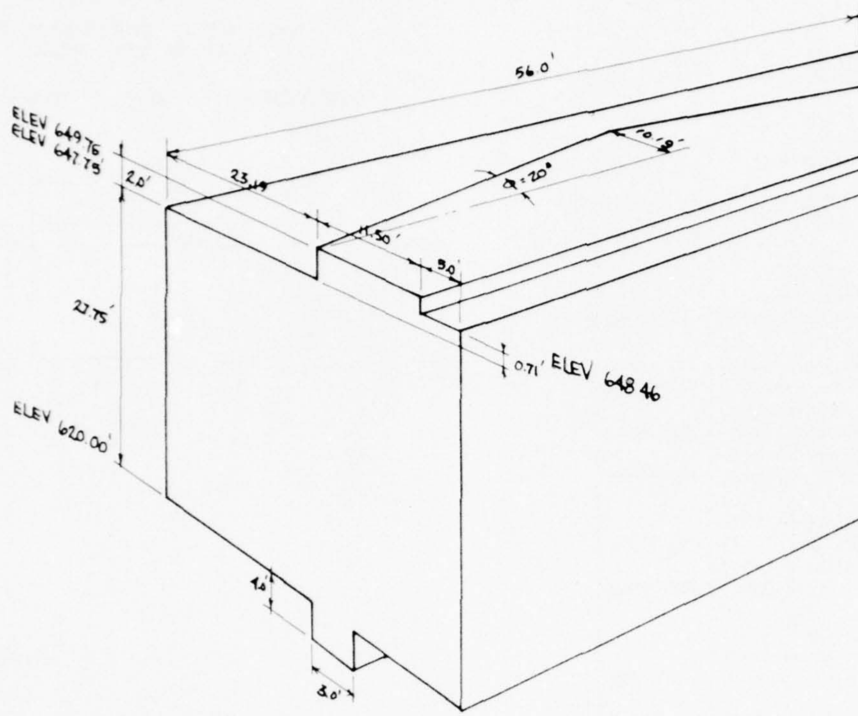


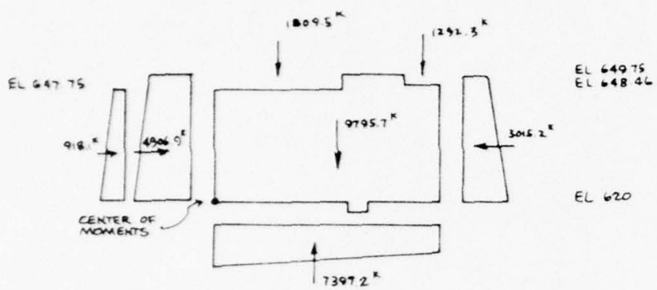
FIGURE A-21 LOWER MITER SILL (56' LOCK)

SUBJECT: LOWER MITER SILL (56' LOCK)

COMPUTED BY: DATE:

CHECKED BY: DATE:

NOT TO SCALE



CASE I NORMAL OPERATIONS

UPPER POOL IN LOCK
LOWER POOL BELOW LOCK

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{CONC}	[.1513][(647.75 - 620)(39.69)(56) + (649.75 - 647.75)(14.5)(56) + (648.46 - 647.75)(5)(56) + (1/2)(649.75 - 647.75)(10.19)(56) + (3)(4)(56)]	9795.7		20.13	197285
$W_{WATER, IN LOCK}$	[.0625][(682 - 647.75)(10)(56) + (1/2)(682 - 647.75)(10.19)(56)]	1809.5		7.84	14186
$W_{WATER, BELOW LOCK}$	[.0625][56][(1/2)(664.5 - 649.75)(10.19) + (664.5 - 649.75)(14.5) + (664.5 - 648.46)(5)]	1292.3		27.39	35396
$P_{WATER, IN LOCK}$	[.0625][56][(649.75 - 620)(32.25) + (1/2)(649.75 - 620)²]		4906.9	13.31	65311
$P_{WATER, BELOW LOCK}$	[.0625][56][(648.46 - 620)(16.04) + (1/2)(648.46 - 620)²]		-3015.2	12.00	- 36182
$P_{EARTH \& CONC, IN LOCK}$	[.1513](.5)(647.75 - 645.75)(56)(645.75 - 620) + [.0754](.5)(645.75 - 620)²(1/2)(56)		918.1	9.60	8814
UPLIFT	[.0625][56][(664.5 - 620)(39.69) + (1/2)(682 - 664.5)(39.69)]	-7397.2		18.75	- 138698
		5500.3	2809.8		146112

$$e = \frac{146112}{5500.3} = 26.56'$$

$$\text{PERCENT ACTIVE BASE} = \frac{(39.69 - 26.56)(3)(100)}{39.69} = 99.24\%$$

$$F.S. = \frac{\Sigma \text{ STABILIZING M}}{\Sigma \text{ OVERTURNING M}} = \frac{301314}{229022} = 1.32$$

FIGURE A-21 LOWER MITER SILL (56' LOCK

SUBJECT: LOWER MITER SILL (56' LOCK)

COMPUTED BY: DATE:

CHECKED BY: DATE:

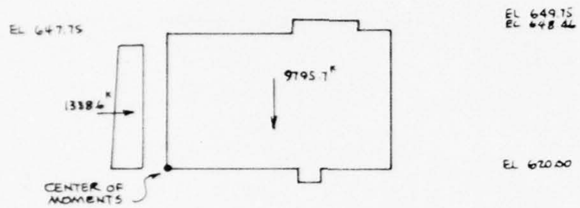
NOT TO SCALE

ITEM	FACTORS	F_V	F_H	ARM	MOMENT
W_{conc}		9795.7		20.13	197285
$P_{earth\ \&\ conc\ in\ lock}$	$[.1513](647.75-645.75)(.5)(645.75-620)(56) + [.1207](1/2)(645.75-620)^2(.5)(56)]$		1338.6	9.28	12421
		9795.7	1338.6		209706

$$e = \frac{209706}{9795.7} = 21.41$$

$$\text{PERCENT ACTIVE BASE} = \frac{(39.69 - 21.41)(3)(100)}{39.69} = 100\%$$

$$F.S. \frac{\Sigma\ \text{STABILIZING M}}{\Sigma\ \text{OVERTURNING M}} = \frac{191603}{12421} = 15.43$$



CASE II MAINTENANCE CONDITION

LOCK DEWATERED

FIGURE A-21 LOWER MITER SILL (56' LOCK)

WES FORM NO. 1253A DECEMBER 1954

SUBJECT LOWER MITER SILL (56' LOCK) | COMPUTED BY | DATE | CHECKED BY | DATE

SLIDING

CASE I NORMAL OPERATIONS

$R = \sum Fv \tan\phi + \text{Key Resistance}$
$= (5500.3)(.5658) + (4)(56)(.075)(144)$
$= 3112.1 + 2419.2$
$= 5531.3$

$SSF = 5531.3/2809.8$
$= 1.97$

CASE II MAINTENANCE CONDITION

$R = (9795.7)(.5658) + 2419.2$
$= 5542.4 + 2419.2$
$= 7961.6$

$SSF = 7961.6/1338.6$
$= 5.95$

BASE PRESSURE

CASE I NORMAL OPERATIONS

$f = \left(\frac{2}{3}\right)\left(\frac{P}{e}\right)/L$

$= \frac{(2)(5500.3)}{(3)(13.13)}/56$

$= 4.98$ KSF

CASE II MAINTENANCE CONDITION

$f = \frac{P}{A} + \frac{Mc}{I}$

$= \frac{9795.7}{(39.69)(56)} + \frac{(9795.7)\left(\left|\frac{39.69}{2} - 21.41\right|\right)(19.84)(12)}{(56)(39.69)^3}$

$= 4.41 + 1.04$

$= 5.45$ KSF

FIGURE A-21 LOWER MITER SILL (56' LOCK)

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ENGINEERING CONDITION SURVEY AND STRUCTURAL INVESTIGATION OF MO--ETC(U)
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1 v. (various pagings) illus. 27 cm. (U. S. Waterways Experiment Station. Miscellaneous paper C-77-2)

Prepared for U. S. Army Engineer District, Pittsburgh, Pittsburgh, Pennsylvania.

Includes bibliography.

1. Condition survey. 2. Dams. 3. Montgomery Locks and Dam. 4. Stability analysis. 5. Stress analysis. 6. Structural evaluation. I. Peatross, James T., joint author. II. U. S. Army Engineer District, Pittsburgh. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper C-77-2)

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